FEB 7 1913

Engineering News

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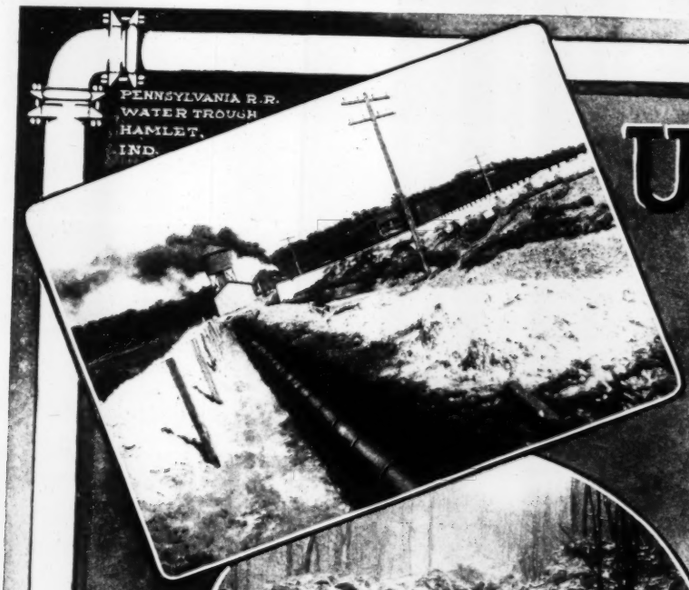

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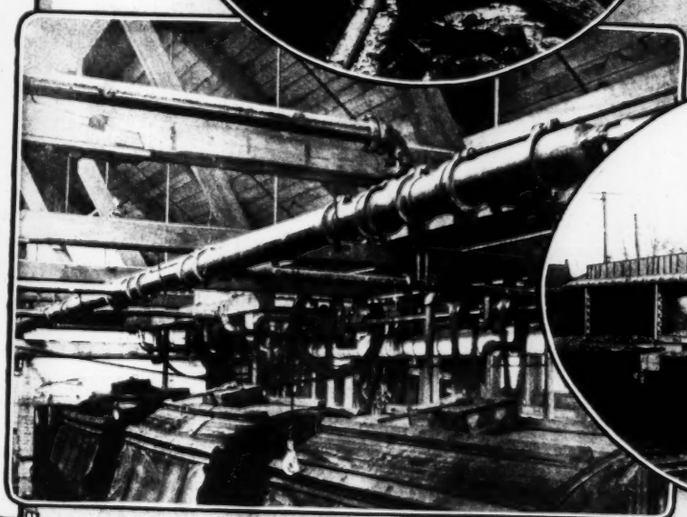
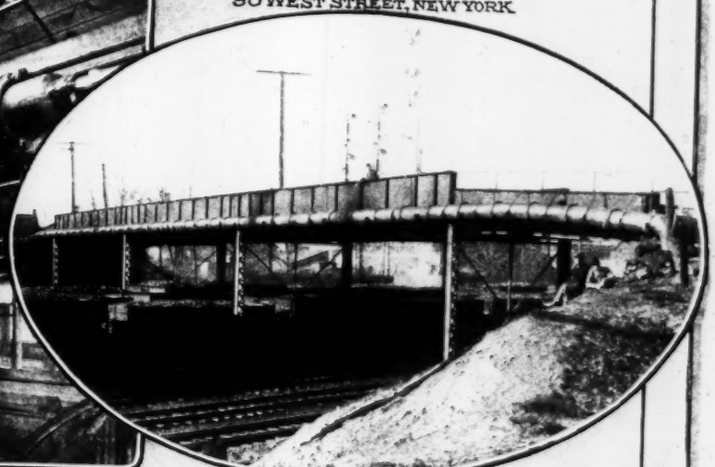


INTERIOR AMERICAN WOOLEN COMPANY PLANT, PLYMOUTH, MASS.



16" PIPE LINE ON BRIDGE AT BAYONNE, N.J.

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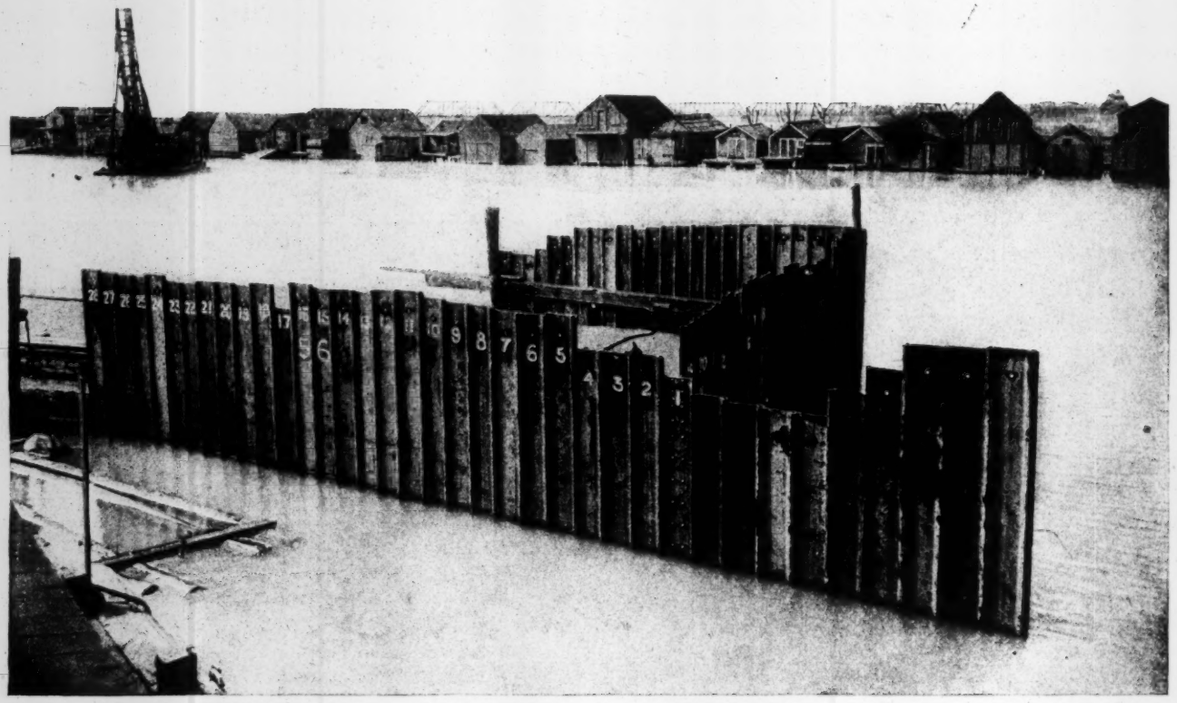

The installation test which proved Lackawanna Steel Sheet Piling superior as to easy, quick, and plumb driving

Above is shown the test installation of Lackawanna Steel Sheet Piling which helped to form the basis for the decision to use Lackawanna Steel Sheet Piling exclusively for the cofferdam in which was built the Government Ship Lock at Black Rock, Niagara River.

Similar installations of five different types of piling in 46 to 50-foot lengths and of approximately the same weights, were made by the contractors, McArthur Bros. & Co., and under Government observation, with the following results.

Section	No. of pieces driven	Total penetration in lineal feet	Square feet penetration	Total number blows	Average number blows per piece
Lackawanna	86	2551.8	2684.75	37267	433.3
Other makes	29	963.4	918.21	13182	454.5
	48	1543.5	1716.38	20838	434.1
	21	713.6	795.38	15612	743.4
	66	2106.5	2742.87	62868	952.4

These figures proved that the Lackawanna section was very economical as to driving, but the splendid condition of the piling structure was even more noteworthy. As shown at the water level in the picture above, the piling was in accurate alignment, and plumb. No difficulties were encountered in making junctions and the corner and special sections were driven more easily than those of other make. In fact Lackawanna Steel Sheet Piling stood first throughout the field test.

Such performance together with the unquestioned supremacy of Lackawanna Steel Sheet Piling in transverse strength and tensile strength of joint assures safety, economy and satisfaction. If you wish to know how Lackawanna Steel Sheet Piling could be used on some particular installation, write us. The advice of our engineers though free is valuable. Ask for literature.

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Engineering News

CHARLES WHITING BAKER, *Editor*

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CIRCULATION OF THIS NUMBER OF ENGINEERING NEWS 20,500

None sent free regularly, no returns from news companies, no back numbers

What The Buyer Needs To Know

Number Six

The buyer needs to know that every advertisement in this paper is "touchin' on or appertainin' to" his business.

Perhaps the buyer—you are one of them, you know—has unconsciously realized that fact without giving it very much thought.

But thereby hangs a tale—yes, two tales.

One has to do with the buyer himself and one is concerned with the history of advertising.

Whatever conserves time and energy is good—a commercial revolution is taking place based on the elimination of waste motion. Whatever saves the buyer waste motion is good—and, on the reverse, whatever doesn't is bad.

This is a business paper. We do not suspect that anybody pays $5.00 a year for it in the hope that it may some day publish R. W. Chambers' latest adventure of the Chamber Maid.

The reader is not asked to divide his attention between a description of the Assuan Dam and the newest method of mending father's socks. He wants it all dam or all darn.

The advertising pages are built along those same lines. No advertising is accepted which is not in line with civil engineering and contracting.

Hence the buyer is not asked to wade through a morass of everything from hairpins to codfish in the hope that somewhere in between he will find something in line with his business.

It saves time, eliminates waste motion to know that he has a paper in which he is certain to find the announcement of every worth-while machine, tool, device, equipment or supply in his business.

The Selling Section is the direct route to what you want to learn about—and no side shows carried.

For some years publishers of high-grade technical papers have adhered to the policy of keeping out the ad. which doesn't belong in the business line of the publications.

Every year many accounts are refused for that one reason.

These publishers know that such advertising will compete for attention with the advertisements which really "belong" in the paper to the latter's detriment.

More, there is a very grave question if it pays to advertise anything outside the line of the paper.

Any publisher in these days who takes advertising which does not give hope of paying has a bad case of mental strabismus.

The illuminating thing about it all is that it was a technical paper publisher who, before any other, decided to keep his advertising columns limited to the field represented by the circulation.

Technical advertising, because of the few dollars it represents compared with the total of advertising, has been referred to by one of the agency gentry as a "small frog in a very large pond."

Strange, isn't it, that most all of the big reforms in advertising have been started by the small frog?

Years after a technical publisher announced a policy of rejecting certain ads., the general magazines followed suit.

With a holier-than-thou attitude they reject the patent medicine ad., the cigarette and liquor ads., and with the other hand reach out for the appropriation of the machinery manufacturer who stands as much chance of making a profitable investment as the soldier's widow did when she put her savings into Ely Central.

Only a short while since one of the general periodicals was making a valiant effort to get $24,000 for a few insertions from a concern making industrial locomotives, concrete machinery and hoisting machinery!

The women, children, clergymen—poor man, beggar man, thief—*everybody* of all sorts and classes subscribe. One per cent. of the circulation *may* be worth while to that particular advertiser but he has to *pay* for it all.

No, Mr. Buyer, you are not asked to share your reading of advertisements in Engineering News with the woes of the heroine, nor are you asked to share your reading of a canal construction article with ads. on Peerless Pants.

Reading pages and advertising are in harmony—the first tells "how," the the second shows "what with."

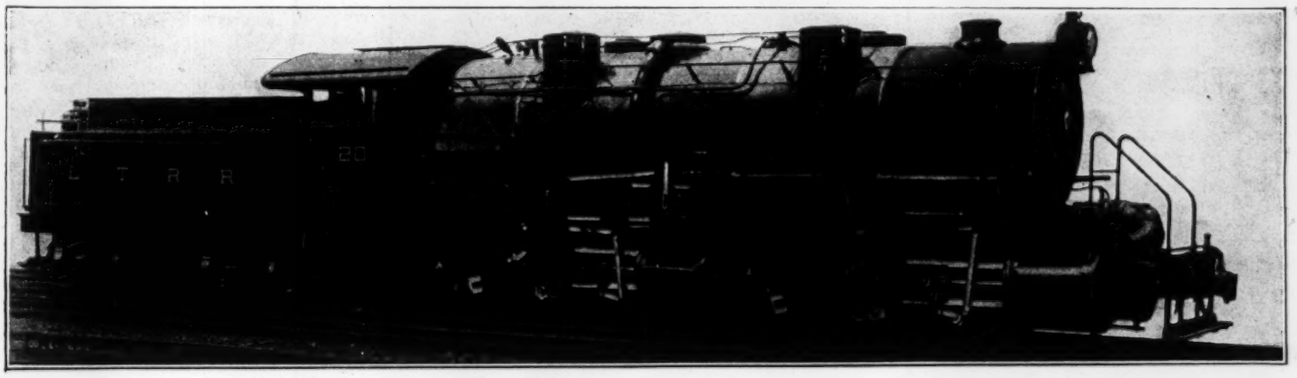
Engineering News

VOLUME 69 FEBRUARY 6, 1913 NUMBER 6

A Heavy Switching Locomotive with Side Sandboxes

On very large locomotives with numerous driving wheels, and especially on Mallet duplex engines, the ordinary sandbox on top of the boiler hardly affords sufficient capacity of sand, as the large boilers leave little height available for the sandbox. For engines in yard and shifting service two sandboxes are commonly used. In some recent Mallet locomotives these boxes are placed on each side of the boiler, so that a much larger box can be used. This arrangement, with two boxes (for the rear or high-pressure group of drivers only), has been applied to large locomotives of the 0-8-8-0 class for the Pennsylvania R.R. The accompanying cut shows an engine of the 0-6-6-0 class having four sandboxes arranged as above described, and serving both groups of driving wheels. The engine illustrated is one used at Lorain, Ohio, by the Lake Terminal R.R., its principal duty being to push trains of loaded ore cars from the ore docks on the lake front to the mills of the National Tube Co. The work is severe, and with the size of boiler required, the distance between the boiler and the clearance limits was too small for a sandbox of adequate capacity. To secure capacity for the required amount of sand, four boxes were placed at the sides of the boiler, as shown, thus enabling large and deep boxes to be used. Each box is fitted with the Leach sanding device. It may be noted, also, that the size of the boiler necessitated placing the bell at one side instead of upon the top of the barrel. In this country the sandbox is placed almost universally upon the boiler, and one box (except on yard engines) serves all the wheels. In European practice, sandboxes are placed usually upon the frames. Very generally they form an extension of the splashers or hoods which cover the tops of the driving wheels, as the engines have the cab floor and running board placed below the tops of the wheels, instead of above them, as in this country. Sandboxes supported by the frames and placed directly in front of the driving wheels, have been tried on a few American engines, but with unsatisfactory results, the sand becoming cold and moist so that it would not flow as freely as when warmed by the heat from the boiler.

The 0-6-6-0 engine shown is of interest on account of the special character of its service. As stated already, it is employed to push trains of loaded ore cars from the ore-receiving docks to the steel mills at Lorain, Ohio. The standard train consists of 21 cars, with a total weight of 1638 tons. While the run is short, the operating conditions are severe. Directly after starting, the engine has to pass a curve of 7° 48′ on a grade of 1.23%, and on straight track there is a maximum grade of 1.65%. The boiler is very large, but has neither superheater nor feed-water heater. The high-pressure steam is taken from a low dome by outside pipes, and the steam distribution of all the cylinders is effected by balanced slide-valves operated by Walchaerts valve gear. The engine was built by the Baldwin Locomotive Works, and its main dimensions are shown in the accompanying table.



MALLET SHIFTING LOCOMOTIVE WITH FOUR SANDBOXES MOUNTED AT THE SIDES OF THE BOILER; LAKE TERMINAL R.R.
(Baldwin Locomotive Works, Builders)

175-TON MALLET SHIFTING LOCOMOTIVE: LAKE TERMINAL R.R.

Class	0–6–6–0
Fuel	Soft coal
Driving wheels (12)	4 ft. 7 in.
Journals	10½x12 and 10x12 in.
Wheelbase, each group	10 ft. 0 in.
Wheelbase, total	29 ft. 8 in.
Wheelbase, engine and tender	62 ft. 3 in.
Length over engine	50 ft. 0 in.
Weight, all on driving wheels	350,000 lb.
Weight of tender, loaded	130,000 lb.
Cylinders (4)	24x32 in. and 37x32 in.
Boiler, straight-top; diameter	7 ft. 0 in.
Boiler, height to center line	10 ft. 0 in.
Barrel plates	11/16 in.
Firebox, radial-stay; size	9 ft. 10 in.x8 ft.
Firebox, depth, front and back	6 ft. 6 in. and 6 ft. 4 in.
Firebox plates; (side, back and crown)	⅜ in.
Firebox tube-plate	½ in.
Tubes, steel; No. 437; diameter	2¼ in.
Tubes, length	21 ft. 0 in.
Heating surface, tubes	5380 sq.ft.
Heating surface, firebox	230 sq.ft.
Heating surface, firebrick arch tubes	30 sq.ft.
Heating surface, total	5640 sq.ft.
Grate area	78.3 sq.ft.
Height to top of smokestack	14 ft. 8½ in.
Water in tender	7000 gal.
Coal in tender	12 tons
Tractive effort (working compound)	82,500 lb.

Lightning Protection of Buildings*

By ERNST J. BERG†

In looking over recent articles dealing with lightning and lightning protection one is struck by the prominence given to protection of electric transmission lines and electric apparatus, while practically no contributions have been made to the subject of protection of buildings since the famous work by Sir Oliver Lodge years ago. Yet the protection of buildings unquestionably is as important as the protection of transmission lines and there are thousands of people interested in protecting their homes to every one interested in a transmission line.

To people living in cities the subject is, however, of little or no interest. Experience has shown that the extensive networks of wires, metal roofs, etc., are usually ample for protection. The man living in the country, however, is very much concerned, as experience has shown that in certain localities at least it is indeed tempting providence not to have some lightning-rod scheme.

While little is known about the source of the electric energy, fortunately, due mainly to the researches of Sir Oliver Lodge, a fair amount of knowledge exists about the nature of its discharge—of lightning itself. Lodge first suggested that there are at least two distinct kinds of discharges; one which is relatively quiet and which results from the gradual breaking down of the air between the object struck and the charged cloud; the other which is a violent secondary discharge caused by a primary discharge in the vicinity.

The first kind follows the well known laws familiar to the electrical engineer—laws that deal with more or less permanent conditions. The nature of the discharge is governed by the resistance, inductance and capacity of the path. The path itself is almost certain to be the rod on account of the conducting streamers above it.

The second kind is more complex and the laws that it follows are less thoroughly understood. There is no conducting path above the rods because there may have been no potential difference between them and the surrounding air before the discharge. Thus the rods may well be missed and the discharge enter any portion of the roof and find its way to ground through the building. To guard against these it would seem that the entire roof should be of metal, or at least largely covered by a metal network.

Professor Fleming has compared the first with the slow combustion of gun powder placed in a room and carefully lighted, the second with detonating powder.

Electrical Constants of Lightning Rods

It is generally rescognized that a lightning discharge is oscillating and that the oscillations are of high frequency, perhaps from 100,000 to several million cycles per second. At these frequencies the electric constants of conductors are very different from those normally.

In the passing of high frequency currents, energy is expended as heat at the surface of the conductor and as electric radiations in the surrounding space.

The object of the lightning rod is to form a path for the discharge, a path offering much less obstruction than any path through the building. If the ohmic resistance were all important this could well be done by using a very large copper rod, but at these very high frequencies the inductive obstruction, or impedance, is measured in tens or even hundreds of ohms, whereas the ohmic resistance may be but a small fraction of an ohm.

Electric Characteristics of Lightning

The greatest number of lightning discharges takes place inside of clouds or between adjacent clouds. These discharges involve usually rather moderate voltages, as has been shown by Steinmetz, but while very interesting are hardly within the scope of this paper. The knowledge of the nature of lightning discharges from cloud to earth is, however, of the greatest importance in studying the efficiency of lightning rods.

Unquestionably such discharges take place not only at moderate voltages, but also at voltages which are exceedingly high; this latter being the case when the charged cloud is separated from earth by a layer of more or less dry, moisture free air.

Under these conditions the distribution of potential may be quite uniform, and the air as a limiting case may be charged to its breakdown point all through its mass, when the potential difference may be exceedingly great, perhaps hundreds of millions of volts. The maximum value of the discharge current may also be very great, reaching several thousand if not hundreds of thousands of amperes, as may be judged from the consideration of the following simple and apparently conservative case.

An area on the surface of the earth 100 ft. square is subjected to the discharge of a cloud or part of a cloud, also 100 ft. square, at a distance of 1000 ft. In this case the capacity is approximately 0.000027 microfarads, a very small capacity indeed, something like that of the smallest Leyden jar. If the electric stress as assumed were uniformly distributed throughout the air space separating cloud and earth, and if the disruptive strength of air is 30,000 volts per cm., a potential of 912 million volts would exist between cloud and earth just before the stroke. The electric charge, that is, the amount of electricity stored, would be 0.025 coulombs, a very small value when considered by itself. The energy stored would, however, be great on account of the high difference of potential. It would be 11,200 kw.-sec., corresponding to the energy of almost a pound of dynamite. This energy must be expended in heat and electric radiation, partly in the stroke before it reaches the rod, partly in the circuit of the rod. The higher the frequency the greater is the relative amount radiated. With a small copper rod the energy radiated at one million cycles is perhaps 50 times as great as that converted to heat in the rod. With a small iron rod, however, the two quantities are not much different. Thus the iron rod will convert more energy to heat and yet only slightly more impede the discharge. The number of oscillations of the current in the iron rod is therefore less than with copper, and the discharge less violent.

While in every discharge almost an infinite number of frequencies undoubtedly are represented, it is probable that one is preponderating. Were it permissible to consider that the frequency in the discharge after it reaches the rod is governed only by the electrical constants of the rod, the wave length would be somewhat more than four times the height of the rod. This would mean with an ordinary dwelling, having a rod of say 50 ft., about 5,000,000 cycles. If on the other hand the effect of the rod is hardly noticeable and the frequency is governed by the distance between cloud and earth, the frequency will be much lower, say 250,000 cycles, with a distance of 2000 ft between the cloud and earth.

In the first case the drop in potential per foot is about two million volts, in the second case only nine thousand volts. The first case, I believe, gives an idea of the conditions of a secondary stroke. It is of very high frequency and may be the result of the discharge of the air immediately surrounding the rod rather than the entire air between cloud and earth. The discharge area is in this case difficult to estimate; it may be quite limited or it may be quite great.

Assuming again an area of 100 ft. square and calculating the voltage and capacity, it is found that the capacity is increased in the same proportion as the voltage is decreased; therefore the charge and maximum value of the current remains unchanged. The maximum value of the current would be, say, 750,000 amp., and the drop per foot of rod about 2,000,000 volts.

It is evident that such discharge would jump several feet in the air rather than travel 1 ft. in the conductor. (The drop in potential of 2,000,000 volts per ft. corresponds to 2 ft. striking distance between parallel planes and perhaps 10 ft. distance between projecting masses of metal).

The second case, I believe, is approached when a lightning discharge takes place from cloud to rod after a conducting path has been prepared by means of streamers. It is the first, the quiet type of lightning mentioned in the beginning of the paper.

Thus is seen how a single lightning rod may be expected to take care of low frequency discharge from cloud to earth, but is entirely inadequate to cope with a violent secondary discharge, even if perchance it hit the rod instead of the building proper.

Were this illustration then at all representative it would mean that one lightning rod, while offering some protection, is entirely inadequate to cope with the situation. If the building were grounded by 10 rods the condition would be much improved. The maximum drop per foot would then correspond to a moderate air space and the lightning discharge would probably be confined to the system of rods.

Before discussing the approximations involved in this elementary discussion it may be well to see the effect of a larger cloud, or perhaps better a larger section of a cloud discharging through the rods. If the areas were doubled, twice as many rods would be required for the same amount of pro-

*Condensed from a paper before the Illinois State Electric Association, Oct. 24, 1912.

†Professor of Electrical Engineering, University of Illinois.

tection. If the areas were the same, but the cloud were only one-half as far above the building, the voltage would be lower, but the capacity greater, and the conditions in the original assumption would apply.

Principal Uncertainties in Theory

The principal uncertainty of this theory lies in the estimate of voltage and frequency. Regarding the voltage, it is not likely that even in perfectly dry air the electric stress is uniformly distributed throughout the space; it is probably higher at the cloud and at the building than in the column of air separating them. The effect of this would be to lower the potential and therefore the charge and energy involved. It is probable that, as stated previously, in discharges in clouds and between clouds the potential may be only moderate because it is likely that the discharge takes place from drop to drop at rather low values. In the case of the discharge to ground, however, this suggestion does not seem reasonable, when the lightning strikes through clear air. Furthermore, unless the voltage is extremely high the energy discharged could not be great enough to do what is frequently done

Regarding the frequency, a great deal has already been said. It seems reasonable that discharges take place at frequencies as low as 100,000 cycles and as high as several millions. While with the former considerably more energy is likely to be involved, it seems reasonable to expect that a rod is of very considerable protection, whereas with the latter a very large number of rods would be required.

Conclusions and Answers

What conclusions can then be drawn regarding the best method of protection; what answers can be given to the questions often asked?

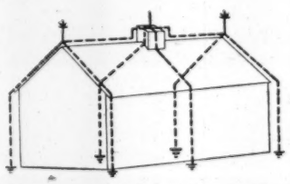


FIG. 1. SATISFACTORY ARRANGEMENT

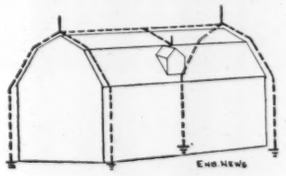


FIG. 2. UNDESIRABLE ARRANGEMENT

FIGS. 1 AND 2. LIGHTNING RODS ON SMALL BUILDINGS

"Is there any real advantage in the installation of lightning rods or do they draw lightning and thereby add to the danger?" Experience seems to have settled beyond reasonable doubt that if properly installed, lightning rods afford considerable protection. A large number of instances might be quoted, but suffice it here to mention only one near at hand. Before equipping the University of Illinois buildings with rods, three fires were caused by lightning; since that time, though the number of buildings has been greatly increased, there has been no damage from lightning.

Any lightning rod "draws" lightning if by that expression is meant that it prepares under certain conditions an easier path for the lightning discharge than would be the case if it were not there. The conducting streamers issuing from the rod tend to equalize the potential between earth and cloud and thus diminish the severity of a stroke and possibly prevent it altogether. Yet it may also be argued that unless there are many such streamers the rod cannot always cope with the situation and a stroke may result from them. This feature is dealt with later under another heading.

"Can a building be perfectly protected from lightning?" The answer must be no, except perhaps in the case of a thoroughly grounded all-metal building.

"How much protection does a lightning rod afford?" Unquestionably some if properly installed, but it may make matters worse in some cases. Assume for instance that a large building is equipped with a high, but broken, rod or a rod having poor joints or a high resistance to ground, say several hundred ohms, which undoubtedly sometimes is the case. Such a rod could serve the function of equalizing the potential between cloud and earth almost as effectively as a good rod, and were there only a sufficient number of them it is conceivable that the neutralization of potential would be so complete as to make a flash discharge practically impossible. A building having one rod only, however, is considered at present. The rod is assumed as projecting considerably above the building. If the electric tension is great, unquestionably streamers are emitted, the air above the rod is made fairly conductive and thus the discharge is invited.

The question is then: "How can such a rod take care of a discharge?" It has been shown how the discharge current frequently is very large and while the ohmic resistance of the rod is practically immaterial as long as it is at all reasonable, it must not approach or exceed the normal value of the impedance. In a rod, say, 30 ft. long, the ohmic resistance of even the smallest practical iron conductor is a fraction of an ohm only and the impedance is perhaps 30 to 75 ohms, depending upon the height and frequency of the discharge. It is easily seen that a poor joint may have many times this resistance; therefore, when the discharge encouraged by the streamers from the defective rod strikes the building it finds the rod entirely inadequate to cope with the situation. The voltage drop in the rod is so great that it is far easier for the current to split up in a number of paths and enter through the building than to confine itself to the rod.

An apparent paradox thus exists. The rod should have good joints, should have good ground connection, and should be mechanically secure against breaking; although the shape of the rod, its metal or general dimensions, are rather immaterial.

"How many rods should be used?" The answer is, the more the better. The protection afforded ought to be, roughly speaking, proportional to the number of earthed rods. Half a dozen ground connections to a house 100 ft. by 50 ft. seems nothing out of the way.

How should they be placed? They should always be placed outside of the building and it is indeed a question whether the vertical part of the system, that is, the rod proper, should not be some little distance from the wall and possibly even insulated therefrom. The rods should be a considerable distance from gas pipes, stove pipes, water pipes and balconies or places where persons might be during a storm.

A small house might be protected as shown in Fig. 1. There are five or six ground connections, one at each corner and one or two from the chimney. A horizontal rod follows the ridge of the roof and is connected to earth by the several ground connections.

Heated gases coming from a chimney are apparently themselves good conductors, or more probable are instrumental in collecting and forming a path of charged particles constituting good conductor for lightning discharges It seems conservative, therefore, to have a metal conductor crossing over the chimney opening. Such a conductor should be made of copper on account of the chemical effect of fuel gases on iron.

Two small spires each having many points are shown. These are unquestionably of some advantage in that through them there is a continuous equalization of the potential between cloud and earth. It seems, however, as if their height should be conservative. A projection of a few feet seems quite enough. To be effective in their office these spires should have several points; but the expense of using platinum or something similar seems hardly warranted, because the amount of current radiated from the points is insignificant before the "brush discharge" begins, and when such discharge takes place it matters little whether the points are rough or sharp.

Each tower or other projection of a building should have direct ground connection, though connected with the other lightning-rod system.

In Fig. 2 is shown what in my opinion is an unsatisfactory arrangement of rods. A small dormer is supposedly protected by an independent short rod connected by a long wire to the main system. It seems unlikely that the discharge would travel many feet to reach the main rod when by striking through the building it can reach ground in most cases with greater ease. The attic room, supposedly protected, is in my opinion endangered by the rod. Protection would, however, result if the dormer rod were directly grounded.

Metal gutters should be connected to the lightning-rod system and should be connected at their lower extreme to ground.

A high factory or power-plant chimney needs good protection; two or three rod do not seem too many and the opening of the chimney should be crowned by some simple copper-rod construction.

"What kind of material should be used?" In cities, where after all the damage by lightning is small, there is always more or less fuel gas and soot in the air and copper seems best. But I can see no electrical advantage in having a stranded cable—a solid rod is as efficient. There is a very slight advantage in a flat ribbon, but the advantage is very small and the mechanical difficulties with a ribbon are greater than with a rod.

In the country, galvanized-iron rods seem best for two

reasons: (1) They are cheaper; (2) their electrical constants are, if anything, better than those of copper. Again an apparent paradox is met. It has been said that the material, size and shape of the rod was rather immaterial and yet now iron rods are recommended. The reason is that the first statement applied to the ability of the rod to discharge the current with the least drop of potential, whereas other factors also enter—factors which, to be sure, are of a secondary importance.

The old idea that the rod carries the electric charge to earth and that that is the end of it is fallacious. The earth is one plate of the condenser, the cloud the other, and whatever energy is stored between them has to be consumed in some way. It is done by heating the rod and by sending out electric radiations. The higher the resistance of the rod the greater is the energy converted to heat, and thus the smaller the number of oscillations of the discharge current necessary to dissipate the energy. Iron has considerably higher effective resistance than copper, and thus has an advantage.

Perhaps the point will be clearer by stating that as far as impeding the flow of the current is concerned the ohmic resistance of the rod is immaterial as long as it is at all reasonable. As far as affecting the duration of the discharge is concerned, however, the higher the resistance the shorter is the time and the less violent is the disturbance. [The resistance should not be concentrated, as at joints, for this would concentrate the energy and destroy the rod.—Ed.].

Lodge's experiments and theory show conclusively that there is no advantage in copper over iron. Copper may have mechanical advantages under certain conditions, for instance in cities where the atmosphere is charged with soot and a varety of fumes. Galvanized iron has the advantage of cheapness, with a possible electrical superiority.

"What should be the shape and size of the rods?" While flat conductors have a very slight advantage, it appears too small to be considered seriously. A round wire or a pipe can conveniently be handled and seems therefore preferable. In the installation of the rods, sharp bends should be avoided as much as possible. There is little or no advantage in using large expensive copper conductors or cables; a size mechanically satisfactory is likely to serve all electrical purposes. Expensive sharp points offer little advantage over ordinary rather blunt points. The rods may advantageously terminate in a number of points projecting only a short distance above the part to be protected.

"How should ground connections be made?" They should be of low resistance, and therefore the rods should preferably terminate in moist soil. "Salting" the ground may be an advantage, but the experience with such grounds is not sufficient to warrant its adoption unless an occasional inspection is made. In many cases excellent connection can be made by driving a galvanized gas pipe a few feet in the ground.

"Should water, steam and gas pipes in the building be connected to the rods and grounded?" This is debatable. On the whole it would seem most conservative to leave them alone, especially the gas pipes. The water pipes, which are always grounded, may, however, advantageously be connected to the lightning-rod system under ground.

"Is it dangerous to stand in a draft during a thunderstorm?" A building having its windows and doors open certainly affords a chance for the entrance of air, perhaps ionized air made conducting by a previous discharge. It is, therefore, safer to keep the house closed during a violent storm.

"Is it safe to stand near a lightning rod?" From the preceding discussion this seems hardly safe. It is well to keep away not only from the rod, but from chimneys, kitchen ranges, metal pipes, etc.

It is finally of interest to draw attention to the fact that the damage by lightning in cities is relatively small and that so far the modern sky scraper with its vast amount of steel appears to be lightning proof.

In conclusion, I wish to emphasize the fact that after all we know little about lightning. I have tried to incorporate the experience of many investigators and observers and am particularly indebted to Sir Oliver Lodge's numerous writings and experiments on this subject.

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Supporting an Electric Railroad on the Edge of a Side Hill

By Walter Loring Webb*

A vexing engineering problem was recently solved in Oil City, Penn., by a method which is probably unique in engineering practice. Colbert Ave. runs along a side-hill slope, which drops down steeply to the tracks of the Pennsylvania R.R., beyond which is the Allegheny River. For a considerable distance the street is about 50 ft. above the railroad and the slope in many places is at or beyond the slope of repose. The street-railway track of the Citizens Traction Co. has been placed at the outside edge of the slope. Immediately following every heavy rainstorm and especially in the spring, when the frost is coming out, there is always a loosening of the soil on the edge of the bank and the railroad has been compelled to use cribbing of poles and old ties to support the outside of their track. Even this cribbing would settle as the surface soil gradually gave way and it was a constant source of expense to even maintain the cribbing and keep up the track to grade. The outer rail especially would settle, causing an outward lurch to the cars which was so alarming that it discouraged traffic. Even if there was never any real danger, the discouragement of traffic and the constant expense required for maintenance justified a considerable expenditure to make the track firm and unyielding.

Naturally, retaining walls were considered. The steep-

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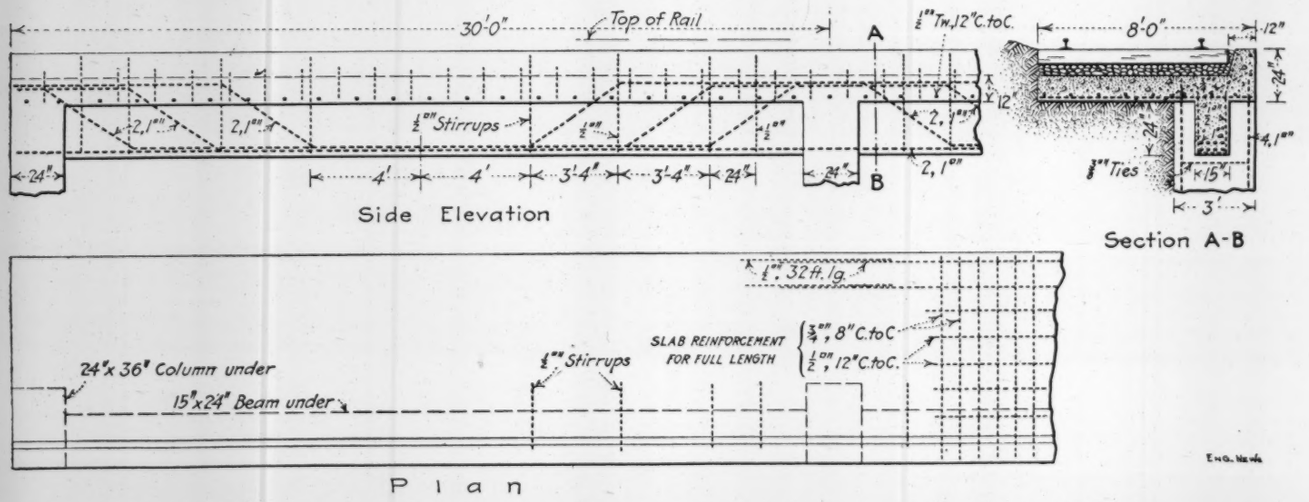

Fig. 1. Details of Reinforced-concrete Slab to Carry Railroad on Side Hill

ness of the side slope would have made it a very difficult problem to support the exposed earth surface while a wall was being constructed, especially if traffic were maintained on the very edge of the slope. A preliminary estimate on the cost of two stretches of retaining wall, one 60 ft. long and the other 120 ft. long, was between $4000 and $5000.

The method adopted has apparently solved the difficulty, has made a stable track and was done at a cost about one-fourth of the estimated cost of the retaining walls. It was argued that if the edge of the slope could be thoroughly supported and the soil underneath protected from frost and weather, the structure would be

FIG. 2. VIEW OF ONE OF THE SLABS DURING CONSTRUCTION

indefinitely solid. A concrete slab 8 ft. wide was built under the track (Fig. 1). Near the outer edge a girder with a depth of 3 ft. and which acted, in conjunction with the slab, as a T-beam, was designed to support the heaviest load of traffic on a span of 30 ft. Two slabs were placed, one of them 60 ft. long and another 120 ft. long, at a place a few hundred feet further on. On the 120-ft. stretch there were five piers 30 ft. apart under the girders; on the 60-ft. stretch there were three piers. The pier pits were dug to a depth varying from 11 to 14 ft. and reached a firm shale several feet below the original surface over which there had been a filling of several feet. The original design called for piers 2x3 ft., but since the pits as dug were not much larger, it was considered preferable to fill them entirely with concrete, thus making a more perfect bond with the soil, saving the use of forms and saving the necessity of backfilling and its tamping. All of the concrete was thoroughly reinforced with steel, the reinforcement being well shown in the view in Fig. 2.

One of the elements of economy in the cost was the lack of necessity for forms. The concrete for the girder was laid in a mere trench. No forms were required for the bottom of the slab nor for its up-hill vertical side. A line of planks 2 ft. high was placed on the down-hill side as shown in Fig. 2, and this was the only planking required, except where a rain-water culvert ran under the slab. Of course, a plank was needed for the inner side of the curb, which was placed after the slab was complete and hard.

Although the soil supported the concrete during construction, it is expected that the soil may shrink away on the down-hill side and thus bring into play the strength of the girder. It is not expected that there will be any appreciable settling or yielding of the soil under the up-hill side of the slab. Even if this should occur to some extent no harm will be done. Considering the slab as a beam lying on its side and having a depth of 8 ft., its strength is enormous and evidently the entire slab would move as a unit if any earth sliding should occur which could move it at all.

It was suggested that the piers might have been extended the entire width of the slab and another girder placed under the up-hill side of the slab. Although this would have made a trestle of the entire structure, which would harmlessly permit the soil to fall away from under it for the entire width of the structure, such a complete failure of the hillside is hardly conceivable. It would have added about 60% to the cost of the work and it was considered needless.

Although the two structures have been in place only since November and it will require a year or two to demonstrate the efficiency of the method, there is every indication that the problem has been solved and that two exceptionally costly retaining walls have been avoided.

The work was done for the Citizens Traction Co., of Oil City, by the Operating Managers, Day & Zimmermann, the work being under the direct charge of the writer.

The Ban Placed On Common Drinking Cups and Common Towels by a number of state boards of health has now been made universal throughout the United States so far as interstate traffic is concerned by an order of Franklin MacVeagh, Secretary of the Treasury, dated Dec. 19, 1912, and addressed to medical officers of the U. S. Public Health Service, and to state and local health authorities:

Common carriers shall not provide in cars, vehicles, vessels or conveyances operated in interstate traffic, or in depots, waiting rooms or other places used by passengers traveling from one state or territory or the District of Columbia to another state or territory or the District of Columbia, any drinking cups, glass or vessel for common use: Provided, That this regulation shall not be held to preclude the use of drinking cups, glasses or vessels which are thoroughly cleaned by washing in boiling water after use by each individual, nor shall it be held to preclude the use of sanitary devices for individual use only.

Common carriers shall not provide in cars, vehicles, vessels or conveyances operated in interstate traffic, or in depots, waiting rooms or other places used by passengers traveling from one state or territory or the District of Columbia to another state or territory or the District of Columbia, any towel for use by more than one person: Provided, that towels may be used again after having been sterilized with boiling water.

Experience with Wood-Stave Pipe in Irrigation

By R. K. Tiffany*

SYNOPSIS—The author gives instructive experiences with wood pipe, mostly composed of staves, used for irrigation purposes on the Sunnyside Unit, Yakima Project of the U. S. Reclamation Service, in Washington. Both wire-wound and banded pipe are reported on. Emphasis is laid on the importance of using a protective coating to cover pipe, joints and collars for pipe laid in dry soil and not kept filled throughout the year. A considerable portion of the article is devoted to leaks in a 55-in. stave pipe, under 190 ft. head, laid beneath a river. Several small leaks in the pipe caused the cutting of the steel bands by the escape of silt-bearing water. After the bands had been cut, the water pressure broke through the end of the unsupported stave. Methods of repair are described.

Owing to its low first cost, the facility with which it may be handled and laid by unskilled labor, the ease with which connections and taps of all kinds may be made, and the belief that its life will equal, or exceed, that of comparatively light gage steel pipe, which is its only competitor in large portions of the arid West, wood pipe has been very popular with many irrigation engineers and large quantities of it have been installed during the last ten years of rapid irrigation development. Unfortunately, but little has been published concerning the various uses of this pipe under the conditions which prevail in the irrigated districts. The following notes are, therefore, submitted with the hope that they may be of interest to irrigation engineers and may result in bringing out some of the experiences of others along the same lines.

The pipes mentioned are all in use at the present time on the Sunnyside Unit, Yakima Project of the U. S. Reclamation Service and were laid by the Reclamation Service, or its predecessor in interest. The irrigation season on this unit extends from Apr. 1 to Nov. 1, and during that time all the pipes mentioned are kept continuously full of water. With the exception of the Mabton pressure pipe, no effort is made to keep the pipes full during the winter season. The Mabton pipe is kept filled by means of a gasoline engine and pump installed at the river crossing for that purpose.

The climate from Nov. 1 to Apr. 1 is cool, with from 4 to 6 in. of precipitation, mostly in the form of rain, and sometimes from four to eight weeks of foggy or misty weather. As a consequence, the pipes do not become entirely dry during the winter, but, on the contrary, the wood is usually found to be moist at any time during the winter, with the exception of a short distance at either end of the pipe line. While the degree of moisture is perhaps not as great as could be desired for the preservation of the pipe, it seems to be sufficient to keep the pipe in fairly good condition, as evidenced by the fact that the decay, in those cases where it has been troublesome, has invariably started from the outside. A few typical installations will be described, with description of surrounding conditions, the behavior of the pipe, and conclusions which have been drawn as the result of our experience.

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Pipe for Lateral Turnouts from Main Canal—In 1893 the Irrigation Co. installed some 40 or 50 pipe headings or turnouts for diverting water from the main canal into lateral ditches. These turnouts were from 18 to 30 ft. long, with a wooden bulkhead and gate at the end next to the canal. They were buried from 4 to 8 ft. below the surface and covered with a puddled backfill of very fine volcanic ash. The pipes were bored from Washington fir, spirally wound with flat iron bands, joined with steel collars and coated with a very heavy coat of asphalt and sawdust. These pipes were removed in the winter of 1911-12, after 18 or 19 years of use, to be replaced by concrete turnouts of larger capacity. The pipes were found to be, almost invariably, in very good condition, with the exception of the two ends, which were occasionally found to be partly decayed. The steel collars were in very good condition, showing very little damage from rust, and the ends of the pipe contained within the collars were, almost invariably, in good order. While these pipes were empty at least five months of every year, the soil surrounding them remained constantly damp and the interior of the pipe never became really dry. The pressure on these pipes in no case exceeded 2 or 3 ft.

Pipe No. 1 was a 4-in. wood-stave, wire-wound pipe, 2000 ft. in length, laid in 1902, to carry water from a concrete reservoir to the headquarters buildings and grounds of the Irrigation Co. at Zillah. This pipe was built with inserted joints, laid at a depth of from 2 to 3 ft. in a very dry volcanic ash. The head varied from 6 ft. at the upper end to about 60 ft. at the lower end. The coating was very thin and light, having more the appearance of ink than that of a good pipe covering. This pipe was kept full of water from the time of installation for about two years, at which time it began to leak and was thereafter emptied during the winter. At the end of four years it was so badly decayed that it became necessary to replace the entire line. It was replaced by a wood-stave pipe, wire-wound, with inserted joints, very similar to the first, but with a very good asphalt coating. The latter pipe has now been in use six years, being full during seven months of each year and empty during the remaining five months. It is in very good order and there are no leaks of any consequence.

Pipe No. 2 is a 13-in. bored pipe of fir, spirally wound with flat iron bands joined with steel collars, and the whole covered with a very good asphalt coating. This pipe, 2200 ft. in length, was laid in 1903 in soil similar to the above, with a covering of 2 to 2½ ft. The maximum head is 25 ft., the minimum 4 ft., and the average about 12 ft. It has been running full for seven months of every year since it was installed in 1903 and when recently inspected, the wood was firm and solid, the bands under the asphalt coating were bright and the collars were evidently in first-class condition. There was no evidence of leakage or decay at any point in this pipe. The material in which it is laid is volcanic ash and the land immediately above the pipe has been irrigated every year since the pipe was laid, the crops being orchard and alfalfa.

Pipe No. 3 is a 14-in. wood-stave pipe, wire-wound, with wooden collars also wire-wound, with a fairly good coating of asphaltum. The length is 5200 ft. This pipe was laid in very dry, sandy soil in the spring of 1908 at a depth of about 2½ ft. No water was carried during the first year and during the three years following the first 800 ft. of this pipe ran less than one-third full. The maximum head is 60 ft., the minimum 4 ft., the average about 15 ft.

In the spring of 1912 it became necessary for the first time to carry a full head in the pipe and the first 800 ft. was found to be so decayed as to be entirely worthless and was taken up and replaced with new pipe. On the balance of the pipe many collars were found to be leaking badly. These were replaced or repaired with wedges, the balance of the pipe being apparently in good condition. In some cases the water was found to be issuing in a jet between the staves of the collar and the staves of the pipe with sufficient velocity that the wires had been cut entirely in two, making it necessary to reband the pipe for a short distance from the joint. The wooden collars, so far as experience on this project goes, have been found to be a source of weakness.

Pipe No. 4 is a 22-in., machine-banded wood-stave pipe, 2300 ft. in length, laid not less than 2 ft. underground in soil similar to the above. The maximum head is 80 ft., the minimum 4 ft., the average about 35 ft. It was laid in the fall of 1906 and it has been giving continuous service since that time for seven months of each year. It has a very good coating of asphalt and has given no trouble, except from occasional leaks at the wooden collars. The collars on this pipe consist of staves about 6 in. long, with two to four steel bands at each collar, provision being made for adjusting the collars by thread and nut on one end of each band. The pipe is apparently in good condition except at the collars, where there have been several leaks, and it has been necessary to renew a number of collars.

The above described pipe and conditions are typical for installations of machine-banded wood-stave pipe in this valley. So far as we have ventured to draw any conclusions, they are substantially as follows:

The one great essential of continuous service from wood-stave pipe that is not kept filled throughout the year is a thorough and complete coating of asphalt on the outside. This coating must cover, not only the body of the pipe, but also all joints, collars or other fittings.

Wooden collars for joints have not given as good results as metallic collars. It has been difficult to secure a tight fit with the wooden collars and the additional thickness of the shell at this point and the break in continuity makes complete saturation of the joints impossible. The result is that decay starts at the joints and from there spreads until a leak develops. Leaks are rapidly enlarged by erosion, especially if the water carries silt or sand in any considerable quantities.

With a good heavy coating of asphalt, and the pipe running full throughout the irrigation season, it appears that absorption of water from the pipe by the surrounding soil is prevented; and well laid pipe under such conditions will probably last 20 years, possibly much longer. With a pipe with poor coating, or empty during a portion of the dry season, decay is very rapid.

MABTON PRESSURE PIPE—This pipe is two miles in length and 55 in. in diameter, carrying approximately 100 cu.ft. per sec., and is of the usual type of continuous-stave wood pipe. The staves are 5½ in. wide, 2½ in. thick, cut to true circle and radial lines, with butt joints made of galvanized-iron tongue driven to a tight fit in slots in the ends of the staves. The maximum head is 190 ft., the minimum 40 ft., the average about 100 ft. Of the total length, about 500 ft. of the pipe is at a depth of from 3 to 6 ft. under the bed of the Yakima River. Fifteen hundred feet of this line, including the portion under the river, has a diameter of 48 in. About 2000 ft. is uncovered, lying on the surface of rocky ground or in rock-cut. The balance of the pipe is covered with soil to a depth of not less than 2 ft., the material varying from volcanic ash to coarse gravel or sand.

There is very little vegetable matter in the soil used for backfilling and, during the greater part of the year, practically no moisture, excepting that which may come from leaks in the pipe. During the winter, however, there is a sufficient rain and snowfall to dampen the ground to a depth of 2 or 3 ft., or possibly more, in loose backfill surrounding the pipe. During at least seven months of the year the soil of the backfill is perfectly dry and, of course, absorbs whatever moisture there may be present in the outer shell of the pipe.

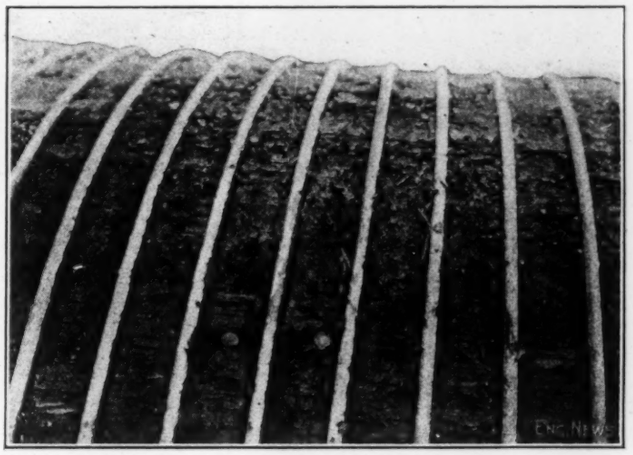

FIG. 1. EXTERIOR OF 55-IN. WOOD-STAVE IRRIGATION PIPE, FOUR YEARS AFTER CONSTRUCTION

(This pipe was covered to a depth of 2 ft. with a soil consisting of fine volcanic ash. The coins shown on the pipe are nickels).

As might be expected, that portion of the pipe lying under the bed of the river is in very good condition and shows no evidence of decay. The exposed portion of the pipe was painted with a red-oxide paint and is now in very good condition, only a slight decay showing at some of the butt joints where there is some slight leakage. The covered portion of the pipe is in fairly good condition, but there are a great many staves which have decayed to a depth of ¾ in. to 1 in. (See Fig. 1.)

Considering our experience with this pipe and the smaller pipe on this project, it would appear that no wood pipe should be used in dry soil of this character without a good protective coating.

BREAK IN MABTON PIPE LINE—The following description of a rather unusual accident and its repair in the above described pipe line may be of interest to those

who are operating or who contemplate the construction of a similar pipes.

The presence of a serious leak in the Mabton pipe was first discovered during the winter of 1910-11, when we undertook to fill the pipe by pumping. The pumping plant installed at the river for the purpose of keeping this pipe full of water consists of a 20-hp. gas engine and a rotary pump with a rated capacity of 450 gal. per min. against a 200-ft. head. Some weeks after the close of the irrigation season of 1910 this pump was started and

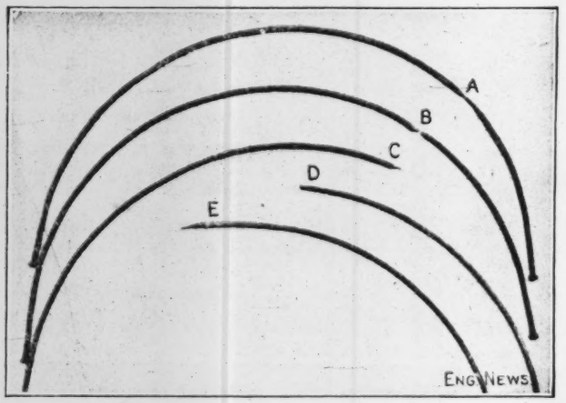

FIG. 2. HOW 5/8-IN. STEEL BANDS WERE SEVERED BY EROSIVE ACTION OF SILT-BEARING WATER UNDER 190-FT. PRESSURE

(These bands were on a 55-in. pipe under 190-ft. pressure, laid beneath a river to convey water for irrigation. See Fig. 3 for view on a larger scale).

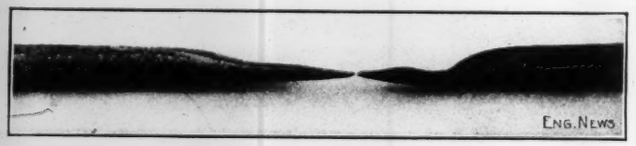

FIG. 3. ENLARGED VIEW OF ONE OF THE SEVERED STEEL BANDS SHOWN IN FIG. 2

water pumped into the pipe until a pressure of about 40 lb. was reached, after which it was impossible to raise the water in the pipe, and it seemed apparent that there was a leakage at that head equal to the capacity of the pump. The entire length of the pipe was carefully patrolled, but no leak could be found on either side of the river, and the conclusion was finally reached that the leak must be under the river. In fact, a buzzing sound, which was apparently caused by escaping water, could be heard from a boat at various points over the pipe; but this sound could not be located, nor was there any evidence of a stream of water sufficiently large to make any disturbance on the surface. Nothing further was done at that time.

The leak was next heard of on the morning of July 29, when the patrolman noticed a drop in the water at the outlet of the Mabton pressure pipe, and on investigation found that a break had occurred in the pipe under the middle of the river crossing at a depth of 8 or 10 ft. under water, from which it was found that about 7½ cu.ft. per sec. of water was escaping.

On Aug. 2 a similar break took place at a point about 75 ft. south of the first, the two together discharging about 12 or 15 cu.ft. per sec. of water.

By sounding with rods and the use of a water telescope, it was ascertained that each leak was small in area and purely local, and not more than two or three bands were broken at each point, but by this time at least one smaller leak had developed and it seemed advisable to take steps to provide a temporary supply of water for the lands under the Mabton Canal in case the breaks should become so large or numerous as to cut off the water-supply entirely.

Accordingly, a trestle was built across the river and 600 ft. of 24-in. wood-stave pipe rushed to the ground for the purpose of providing a temporary crossing if found necessary. Water service, however, was continued through the pipe and as the leaks did not increase appreciably, the pipe was continued in service until Oct. 1, when steps were taken to effect, if possible, a permanent repair.

The service of a diver were secured and it was found that the two large leaks were very similar in character, being roughly triangular in shape, with an area of 16 to 24 sq.in. each. Each break had occurred at the end of a stave and at each point two of the steel bands were found to be entirely severed and a third almost entirely severed by the erosive action of the water and the silt and sand which it carried. A third smaller leak which was found apparently shows the beginning stage of the larger leaks. At this third leak a small hole was eroded at the end of a stave, the water apparently following around the metal tongue and a band being already slightly eroded, and had this not been repaired in time it seems probable that a break similar to the other two would have occurred in the near future. It appears that the erosion cut one or more bands, and the end of the stave was then broken off by the water pressure. Figs. 2 and 3 are views of the severed bands.

The breaks were repaired by placing plates of 3/8-in. steel inside and outside of the pipe at each break, the inner plate having its smallest dimension equal to the largest dimension of the opening in the pipe, both plates being curved to fit the pipe, with rubber gaskets between the plate and the staves. These plates were drawn together by a 3/4-in. bolt through their centers. New bands were placed where the old ones had been destroyed, passing over the outer plate and then cinched up as tight as possible.

After the job was finished and before the diver had left, a full head of water was turned into the pipe again and the pipe carefully sounded and inspected throughout its entire length. Under the river, at one of the places repaired, a very small leak appeared, but this was entirely stopped by placing several sacks of sawdust and alfalfa meal in the pipe through a manhole at the north end of the river crossing, and then turning in a small head of water.

The entire cost of the repairs made by the diver, including all labor and material, was $1260. The greater portion of the time was spent in removing the material from around the pipe, so as to be able to get at the band connections and to get a sufficient room to work in placing the patches. Several plans were tried for removing the material from around the pipe, but it finally became necessary to install a 6-in. sand pump on a scow, which was brought up the river from Prosser. The gasoline pumping engine from the pump house was moved

aboard the scow to drive the pump and the diver handled the suction hose, and by this means the material was removed very rapidly after the pumping plant was once in operation.

It is a noticeable fact that these three leaks all occurred in the upper third of the pipe and I have also noticed that there are numerous very small leaks in the upper third of the exposed portion of the pipe south of tf the river, and practically none in the lower two-thirds of the pipe. This may be due to the fact that the velocity through the pipe up to the present time has been only 2 to 4 ft. per sec., and throughout the length of the pipe a certain amount of fine silt has been deposited on the sides and bottom. This would, of course, have a tendency to fill all small leaks in the sides and bottom, but probably not those in the top. It is, therefore, proposed to feed into the pipe from time to time a few sacks of fine sawdust, bran, alfalfa meal, or something of a similar character for the purpose of plugging any small leaks before erosion sets in.

An 18-in. manhole has been placed near the river margin at each side of the crossing and it is believed that similar leaks may be stopped, should they occur in the future, by pumping the water from the pipe under the river and making repairs on the inside of the pipe. It appears that repair may be made in this way, provided the leaks are discovered before any bands have been destroyed, and with our experience of the past year, we should be able to anticipate and prevent any serious breaks.

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Sheet-Pile Cut-off for the Protection of Levee Foundations

By C. O. Sherrill*

The readers of Engineering News are, no doubt, familiar with the principal conditions under which levees are built, the most notable of which is that their location is governed by other conditions than the suitability of foundation or materials for their construction. Unlike most other engineering structures, no extensive preparation of foundation is usually possible on account of the extent of the work and the relatively small amount of money available; the earth found in front of the levee must be used for construction, even though known to be wholly unsuited for the purpose.

When a levee is thus built of inferior porous material, the usual method of strengthening it is to increase the area of cross-section. For defects in the levee proper, this increase of section by the addition of earth is usually best because it is the cheapest means of adding strength; but if leaks develop deep down in the foundation and show up as sand boils as far inland as 600 ft. from the levee, no satisfactory remedy has been found.

A leaky levee of small section may also be protected by concrete revetment, especially where the levee is subject to wave wash. This method has been used for several years by the Fourth District, Mississippi River Commission, with entirely satisfactory results. On the river face of the levee, a weak concrete without reinforcement is laid with its toe resting on a vertical curb extended for 3 ft. below the base (Fig. 2). This revetment has been found to be an effective remedy against wave wash and seepage.

Sand boils are large springs often 6 or 8 in. in diameter, out of which water and sand flow under the pressure of the water on the river side of the levee, whose surface is from 10 to 20 ft. above the base. These sand boils are harmless unless they carry a large quantity of sand, but when the flow of sand becomes excessive, indicating rapid erosion of the base, they are a constant menace to the safety of the levees and require immediate attention to prevent disaster. A large percentage of crevasses, except where due to overtopping, have been caused by sand boils, which appear to be the result of filaments of water flowing in horizontal porous strata until they unite in one stream and burst through to the surface at the point of least resistance. To cut off this underground flow, muck ditches have been used, but with indifferent success, because of the prohibitive cost of digging to a sufficient depth and the impossibility of securing good clay filling in the vicinity. To overcome the difficulty, the Fourth District, Mississippi River Commission, is now having 5000 lin.ft. of levee at White Oak Lake, 35 miles below Vicksburg, on the west bank of the Mississippi, protected by means of dove-tailed sheet-piling (Fig. 1) driven 20 ft. deep at a point on the levee 4 ft. above the river toe. About 5000 ft. of levee at Grand Bay, 10 miles north of Baton Rouge, La., will be protected in the same manner. If the sheet-piling at these two localities proves successful during the next high water, this method of protection

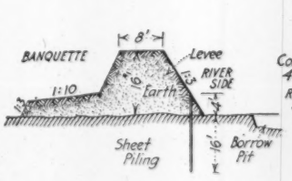

Fig. 1

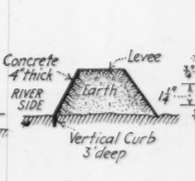

Fig. 2

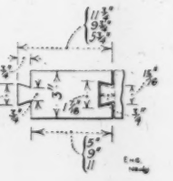

Fig. 3

will be extensively used on the Mississippi River Levees in future.

The sheet-piling used is of No. 1 common heart cypress and it cost, machined, as shown in Fig. 3, $29.50 per M. delivered on U. S. barges at Plaquemine, La. Heart cypress was selected as it lasts indefinitely in damp earth; pine would be easier to drive, not being so brittle, but does not last so well. The dove-tailed section gives a water-tight joint when the timber is expanded to a close fit in the damp earth.

The contract for driving the piling was let to J. B. Alexander, New Orleans, La., at $1.87 per lin.ft. of levee protected, or $0.0935 per sq.ft. of piling. The work advanced slowly at first, on account of breakdowns, but at present an average of 200 piles are driven per day.

Some doubt was felt as to the possibility of driving piling only 3 in. thick to a depth of 20 ft. on account of the small area of cross-section. It has been found that a single pile 3x6 in. cross-section has not enough strength to resist the blows from an 1850-lb. hammer, so the plan of threading two piles together and driving this double pile was tried successfully. An average of 55 blows is given each double pile. A small number of piles have been broken or split, but in general about 6 in. of brooming of the end of the piles is the only injury from driving. The driving is comparatively easy except in strata of compact sand.

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The V-Notch Weir Method of Measuring Water*

By D. Robert Yarnall†

Attempts to measure accurately and to record automatically the flow of water through pipes or channels have been made from time to time with varying results as to accuracy and convenience. This paper will attempt to show results obtained by the simple and practical V-notch method of measurement, which though not new, seems now to be meeting with marked success, both abroad and in America, in power plant measuring problems. Tests of the accuracy of this method of measurement have recently been made in America and the data collected are here presented to show the degree of accuracy attainable.

The instrument employed in the tests is the Lea V-notch recorder. An apparatus which depends on the laws governing the flow of water through weirs. These laws have been found to work with extraordinary accuracy but it is only comparatively recently that they have been turned to

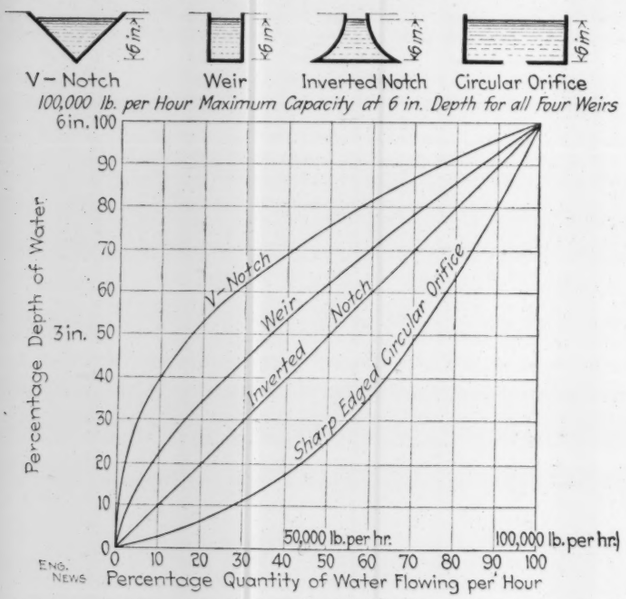


Fig. 1. Curves Showing Comparative Accuracy of Different Types of Weirs under Different Rates of Flow

account in measuring the widely varying rates of flow, both hot and cold, met with in power plants.

The most common forms of weir are the plain horizontal sill and the sharp edged rectangular notch. The former of these has the full width of the stream or channel and is without end contraction, the formula used in connection with it being that of Francis:

$$\text{cubic feet per second} = 3.33\ LH\sqrt{H}$$

where

L = The width of the weir in feet.

H = The head of water passing over it in feet.

The latter is a sharp edged rectangular notch of less width than the channel or stream and therefore has end contraction. The formula used in this case is

$$\text{cubic feet per second} = 3.33\ LH\sqrt{H \times C}$$

where

L and H are the same as before.

C = a constant which depends on the ratio of the width of the weir to the width of the channel.

Although these two formulas give a fair degree of accuracy, there is another in which the accuracy is very nearly 100 per cent. This is Prof. James Thomson's formula for the sharp edged V-notch weir:

$$\text{cubic feet per minute} = 0.305\ H^2\sqrt{H}$$

where

H = The height of the notch in inches, the angle of the notch being 90°.

Table I gives the flow of water through 90° V-notches for each inch of head, up to and including 15 in., calculated by Thomson's formula.

TABLE I. FLOW THROUGH 90° V-NOTCH WEIRS

Depth in notch, in.	Flow per hour, lb.	Depth in notch, in.	Flow per hour, lb.
1	1,140	9	277,960
2	6,480	10	361,740
3	17,830	11	459,030
4	36,610	12	568,720
5	63,940	13	694,710
6	100,860	14	836,110
7	148,290	15	993,510
8	207,060		

The section of the flow through the V-notch is at all times the same shape, though the area may vary, and this constancy of form tends to simplify the formula and make it accurate. This form of notch is also especially adapted for measuring smaller quantities of water than the rectangular wiers, as shown by the curves in Fig. 1 which give a comparison of the accuracy of the different types of wiers at different rates of flow. Another property of the notch is that its angle may be less than 90° without impairing its efficiency, which enables it to be used to measure very small quantities of water.

It was for these reasons, as well as for its accuracy, that the V-notch was adopted for use in connection with the Lea recorder, a sectional view of which is shown in Fig. 2. It will be seen that a float spindle, rigidly attached to a float which rides on the surface of the water flowing over the V-notch, passes up through the bottom of the instrument case. A rack on this spindle engages a small pinion upon the axis of a drum revolving between centers. Upon the body of the drum is a screw thread, shown in Fig. 3, the contour of which is the "curve of flow" for the V-notch in connection with which the recorder is used, and as the flow through the notch increases rapidly with the depth, the pitch of the screw increases in the same proportion. Above this drum as shown in Fig. 4 is a horizontal slider bar, supported on small pivoted rollers and carrying an arm, at the upper end of which is a pen or pencil point in contact with a paper chart upon a clock-driven recording drum, which revolves once in 24 hours. As the float rises or falls the drum spiral is rotated, and its motion is imparted to the slider bar and pen arm by means of a saddle arm projecting from the latter and engaging at its lower end with the screw thread. On the body of the spiral drum is a graduated scale adjacent to the screw thread and computed in pounds per hour. It is thus seen that the drum serves the double purpose of rectifying the motion of the pen so that it moves equal distances for equal increments in the rate of flow, and of providing a magnified scale for making an accurate observation of the rate of flow at any moment.

The apparatus has the following advantages:

a. It is simple and hence dependable.

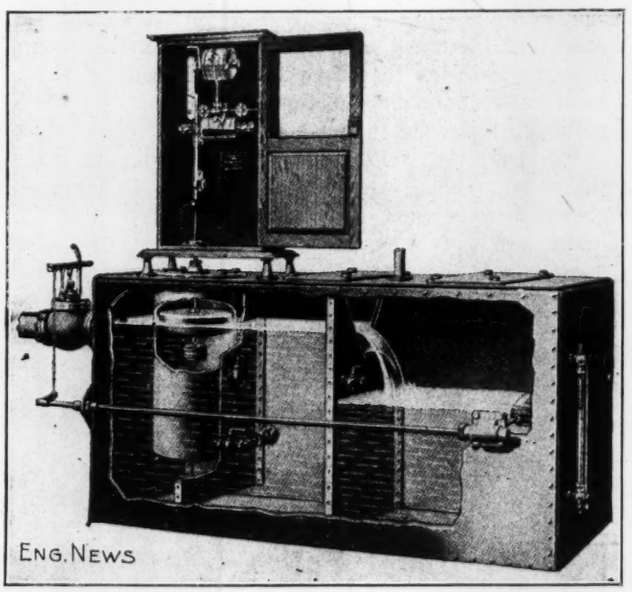


Fig. 2. Sectional View of V-notch Weir Meter with Lea Recording Mechanism

*From a paper read before the American Society of Mechanical Engineers at its annual meeting in New York City, December, 1912.

†Mechanical Engineer Yarnall-Waring Co., 1109 Locust St., Philadelphia, Penn.

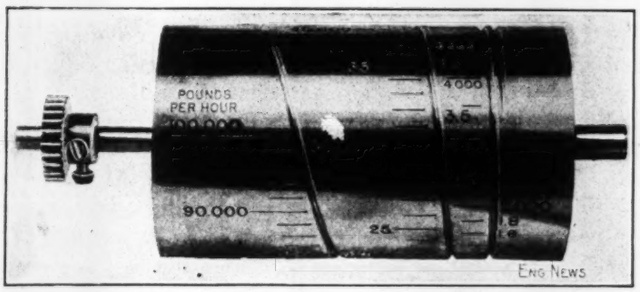


FIG. 3. RECORDER DRUM, SHOWING GRADUATED CURVE OF FLOW

b. Its operation is continuous.

c. There are no moving parts in contact with the flow except the float, which is in a chamber by itself and not in the path of the flow.

d. Collections of sediment and deposits of scale do not affect the accuracy.

e. Owing to change of density of the liquid being measured, due to changes in temperature, the float which actuates the recorder is immersed more or less, depending upon this change of density, and the recorder and the float are so constructed that they compensate automatically for these changes in temperature over a wide range.

f. By making the tank of ample proportions a considerable interval of time takes place between changes in the head on the weir, so that the recording apparatus has a chance to adjust itself to changes in the rate of flow, thus giving a chart very easy to measure with the planimeter.

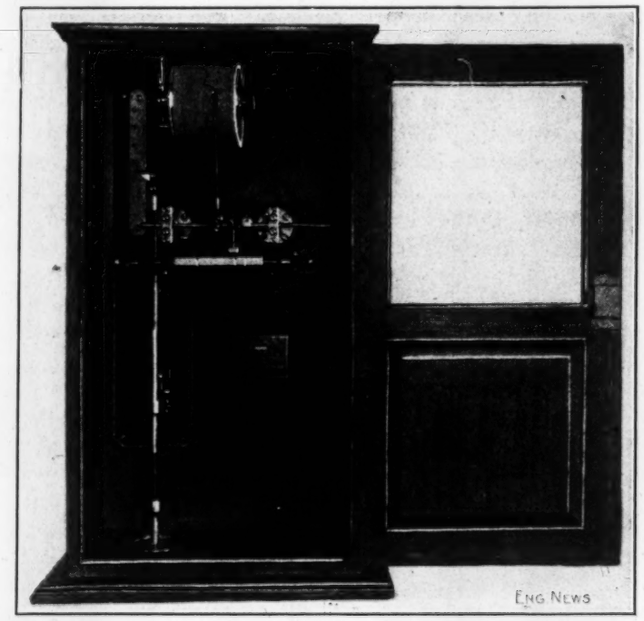


FIG. 4. ENLARGED VIEW OF LEA RECORDING MECHANISM

g. It is always placed on suction side of pump, hence it is available at all times for inspection, cleaning and repairs.

Like the Venturi meter and the Pitot tube, the V-notch recorder is a continuous flow meter, but unlike these it is not dependent for its accuracy on velocity changes, and hence its record is not subject to the fluctuations due to sudden changes of velocity head observed in these meters while measuring rapidly changing rates of flow. Perhaps the greatest advantage of this method of measurement is found, however, in the way by which it may be checked independently of the recording mechanism. The operator may construct alongside of his recorder an independent "hook" gage or use the graduated vertical scale and pointer on the extended float rod in the instrument, and can observe from time to time the number of inches head on the weir. By substituting these values in Thomson's formula, the rates of flow as shown on the recorder chart may be checked quickly and accurately. Thus the operator need not depend upon the recorded result until he is sure that it agrees with the figured result from his hook gage readings. It is believed, therefore, that the V-notch with its tank is the most satisfactory testing set that could be devised for measuring water under varying conditions of temperature, continuity of flow, and rate of flow.

The tests mentioned in the opening paragraph of the paper were made at Dartmouth College, Philadelphia and New York City. At the first location there were two tests, one in the winter and one in the summer, conducted by the engineers of the college, Messrs. R. D. Kimball Co. The graphic charts of these two tests are shown in Fig. 5, where the top chart shows the record of the winter test, the middle chart that of the summer test, while the record of the Philadelphia test is given the lowest chart.

The winter test at Dartmouth College was made at the normal running load and took care of all the natural fluctuations in the returns from the heating system, which was being taxed to its utmost by an outside temperature only slightly above zero. These fluctuations were due in part to poor control of the return water from the heating pipes at the new Gymnasium, and in part to the small water-storage capacity of the receiver. The summer test was later run with the same apparatus as the first test, but the rate of flow and amount of fluctuation were greatly reduced because the Gymnasium heating system was out of service. The Philadelphia test was made with water of considerably lower temperature and much higher rate of flow. Also in this case the amount of fluctuation was comparatively negligible. The tests at New York were of longer duration and employed a meter of the larger capacity used.

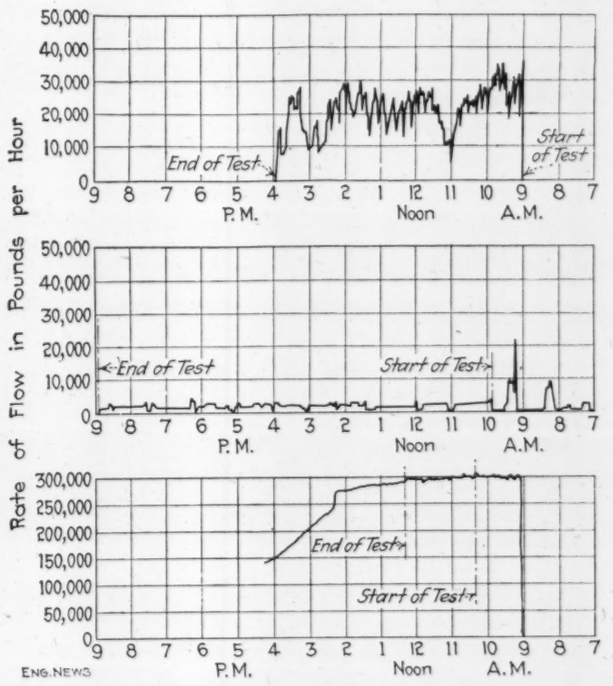


FIG. 5. RECORDS OF TESTS ON V-NOTCH WEIR METERS

In Table II is shown the comparison of the principal data obtained in these three sets of observations and from this table it is apparent that the V-notch weir method of measurement, using the Lea recording mechanism, can be depended upon to within about 1%, even in extreme cases of fluctuating character of flow and changing temperature.

TABLE II. RESULTS OF TESTS ON V-NOTCH WEIR METERS

Location	Dartmouth Col.		Phila.	New York City				
Date (1912)	3-23	8-2	8-9	4-19	4-20	4-22	4-23	4-24
Duration of test (hr.)	6.25	10.2	1.96	5	8	13	8	8
	Var.	Var.	Var.					
Temperature of water (F)	160°-195°	160°-195°	72°-74°					
Corrected weight of water by tank (lb.)	143,841	41,625	596,867	291,965	266,009	557,974	352,725	352,725
Corrected weight of water by meter (lb.)	145,538	4,1395.7	602,270	290,525	267,260	557,775	351,501	352,231
Error of meter (lb.)	+1697	—229.3	+5403	—1440	+1251	—199	—1224	—494
% error of meter	+1.18	—0.55	+0.9	—0.49	+0.46	—0.03	—0.34	—0.14

—=low reading. +=high reading.

Annual Convention of the Illinois Society of Engineers and Surveyors

The 28th annual meeting was held Jan. 22 to 24, at the Great Northern Hotel, Chicago, with the President, J. A. Harman, in the chair. The attendance was good, and there was such a long list of papers that it was necessary to read some of these by title. The entertainments included the annual dinner (on the subscription plan) and excursions to four points of interest: 1, the Chicago Ave. pumping station of the water-works system; 2, the pumping and sewage-testing and experimental plant of the Sanitary District of Chicago at 39th St.; 3, the new Milwaukee Ave. viaduct, which has plate girder, truss and reinforced-concrete spans, and, 4, the boulevard system of the South Park Commission, showing different kinds of paving, public playgrounds and ornamental concrete work. At one of the meetings, papers were presented describing the several works to be visited. A short address was made also by Wm. C. Fraser, President of the Minnesota Surveyors and Engineers Society. At the dinner an address was made by S. S. Greeley (a Past President of the Society and one of the pioneers among Chicago surveyors), and a lecture on "City Planning" was delivered by Geo. C. Hooker, Civic Secretary of the City Club. This was accompanied by a number of lantern slides illustrative of work and conditions in this country and abroad.

The address of the President, J. A. Harman, was of a rather unusual character, taking for its subject the weakness of our present educational system (from the public schools to the universities and technical colleges) in preparing youth for the actualities of commercial life. He contended that the word "education" has come to be considered as "book learning" and that as now practiced, it fails to give an adequate idea of or preparation for future responsibilities. Manual-training systems are largely "playing at work," which is one reason why the interest in them is comparatively small. He suggested that the time should be devoted partly to general education, partly to the competitive production of something useful, and partly to pleasure and recreation. The report of the Secretary, E. E. R. Tratman, showed a membership of about 230 and a cash balance of about $1100.

Water-supply and Sewerage—The report of the Committee on Water-works, presented by D. H. Maury, reviewed some of the developments in the pumping, storage and purification of water, and a paper by Paul Hansen (Chief Engineer of the Illinois State Water Survey) discussed the advantages of expert control over the operation of public water supplies and sewage-purification works. The pumping stations of the Chicago water-supply system were described by H. S. Baker. Diagrams of rainfall records of Illinois were submitted by G. C. Habermeyer (Urbana), and in relation to these was a report by M. W. Hayes (St. Louis, Mo.) on a phenomenal rainfall at Alton, Ill., in July, 1912; this record was 9.22 in. for a fall of very short duration. The sewage-testing plant of the Sanitary District of Chicago was described by Langdon Pearse, but nothing was said as to the relation of these investigations to the future policy of the Sanitary District as affected by the recent decision of the Secretary of War, denying permission for an increased supply of water from Lake Michigan to the drainage canal. An interesting paper by S. A. Greeley (Winnetka) discussed the subject of garbage collection and disposal for cities of the smaller class. Illinois has 30 cities of over 10,000 population (exclusive of Chicago). The three important matters to be considered are the house treatment, the collection and the disposal. The reduction system of disposal is not generally in use in cities of less than 100,000, although one at Vincennes, Ind., serves a population of 15,000. Other methods are burial, feeding to pigs and burning. The last is the best for cities of 20,000 to 30,000, while the others may be satisfactory in the smallest cities. The cleaning of a small septic tank that had become clogged and foul through neglect was described by L. Z. Jones, City Engineer of Galva, Ill.

Surveying—The meeting was rather notable for the amount of attention given to the subject of surveys and surveying instruments, and the exhibits also included an extensive display of such instruments. A paper by D. D. Scott (Rochester, N. Y.), dealt with a new precise Y-level. Some recent improvements in engineering instruments were described by E. E. R. Tratman (Engineering News). The methods of making topographic surveys for drainage district were dealt with in a paper by W. A. Birket (Peoria), and a paper by A. L. Webster (Wheaton) described river surveys made on the ice on account of the swampy condition of the country making it almost impossible to do the work in any other way. The methods of surveying for partition of lands were discussed by E. P. Montblanc (Chicago). The report of the Committee on Surveying touched upon the vexed question of the county surveyor; his qualifications, and his duties, as they are and as they should or might be. The land surveyor, as distinct from the engineer or engineering surveyor, is far from being extinct, and he seems to have greater political ability and activity than the engineer. In an address, S. S. Greeley (Chicago) a pioneer surveyor of wide experience and high repute (whose experience dates back to the time when there was very much more work in surveying than in engineering), humorously characterized the surveyor as "an engineer in an incomplete state of development."

Land Drainage—This is another subject of very live interest in the central West, and one which always receives a good share of attention at such meetings as this. In addition to papers noted above which related more or less to the subject, there was a paper by C. A. Sheppard (Edwardsville) on "Drainage Projects in Southwestern Illinois," and reports were made by committees on drainage and drainage legislation. Somewhat related to this subject also was a paper on "The Floods in the Mississippi River," by A. C. Dabney (Memphis, Tenn.), which presented a review of information given by the author in Engineering News a few months ago.

Roads and Streets—Nearly every kind of paving received attention during the part of the meeting devoted to this subject. These included brick, concrete, stone, wood-block and bituminous paving, for city streets, park roadways and country highways. Apart from papers relating to paving proper, there was a paper by A. N. Johnson (State Highway Engineer) on the work of the Illinois State Highway Commission; another on "Modern Road Machinery and Its Work," by T. R. Agg (Springfield, Ill.), and a third on "Cost Data for Paving Construction," by W. W. Kerch (Granite City). A topic

suggested for discussion was, "How to make concrete sidewalks that will last," but nobody seemed disposed to offer suggestions as to this.

MISCELLANEOUS—Four papers dealt with structural design and construction work. These included "The Track Elevation Work of the Chicago & Alton R.R. at Joliet, Ill.," by R. A. Cook (C. & A. R.R.); "Specifications and Design-Standards for Reinforced-Concrete Structures," by E. McCullough; "The Milwaukee Ave. Viaduct," by L. K. Sherman, and "Novel Construction of Concrete Foundations," by W. G. Kirchoffer. This novel method (for soft ground) consisted in building the work in sections which were sunk to place by their own weight, requiring only a minimum of form work. The possibilities of "Park Development in Small Cities" were presented by A. E. Nelson, and Prof. H. H. Stoek (University of Illinois) outlined the coöperative work on mining in Illinois which is being done through the joint service of the U. S. Geological Survey, the U. S. Bureau of Mines, and the Illinois Geological Survey.

OFFICERS—The officers for 1913 are as follows: President, L. K. Sherman, Chicago; Vice-President, J. J. Harman, Peoria, Ill.; Secretary, E. E. R. Tratman, Wheaton, Ill. The next meeting will be held at Peoria, Ill., in January, 1914.

Annual Meeting of the American Society of Heating and Ventilating Engineers

The 19th annual meeting of the American Society of Heating and Ventilating Engineers was held at the Engineering Societies' Building, in New York City, on Jan. 21, 22 and 23. There were five sessions for the presentation of papers, reports, etc., in addition to a banquet and various excursions for the guests.

The following officers were elected by a count of letter ballots: President, J. F. Hale, of Camden, N. J.; Vice-Presidents, E. F. Capron, of Chicago, and A. B. Franklin, of Boston; Treasurer, J. A. Donnelly, of New York.

By a change in the organization no secretary was elected, the office now being appointive. A "council" also replaces the "board of governors."

Various committee reports were received, such as the committees on compulsory legislation, standards of ventilation, heating guarantees, tests, and various conferences. Throughout the meetings, there was an evident desire on the part of some members that these reports should result in "standards of performance," or "recommended practice," etc. Some members, on the other hand, wished to postpone action always to some distant convention, or to have some committee continue to deliberate. The net result was a succession of parliamentary tangles, which used up time that was needed for engineering discussion.

Of all these reports that which aroused the most interest probably was the one on compulsory legislation for heating and ventilating. The chairman of this, Prof. J. D. Hoffman, of Lincoln, Neb., made a brief progress report by mail, showing how the committee was at work in its endeavor to prepare a model bill which they might recommend for passage in any states that were discussing the advisability of compulsory ventilation and heating. The committee divided the model law into two parts, one to provide for schools, hospitals and amusement places, etc., and the other particularly drawn to meet conditions in factories, etc. A tentative draft was submitted on the first division only.

Several speakers deplored the years of delay in the various committees on this subject, and asked that the present one push its work more vigorously so that within the next few months a model law would be in shape for recommendation to all inquirers. In the discussion of the merits of the specific recommendations, there was criticism of imposing any rigid requirements for sick rooms, humidity figures for factories, etc.

A committee on a ventilation standard for motion-picture theaters brought in a model law which they desired the society to adopt for recommendation in all instances where advice was sought as to the requirements which should be imposed on these amusement enterprises. The ordinance embodied certain minimum requirements, such as a floor area of 4.3 sq.ft. per occupant exclusive of aisles, entrances, etc.; 80 cu.ft. of air space per occupant for the hall; a positive supply of outside air, furnishing at least 15 cu.ft. per min. per occupant; a temperature between 62 and 70° F., except when no heating should be needed; a maintenance of temperature and ventilation without drafts; an exhaust ventilation of 60, 90 or 120 cu.ft. per min. from the picture-machine booth, depending on whether one, two or three machines were installed; and a fireproof vent pipe from the machine booth. Gas radiators were prohibited from installation in the theater. The report was received and ordered printed as information.

By request, a committee was appointed to bring in for discussion a recommendation of methods of using an anemometer in front of an air register. Finally it was agreed that where an anemometer had to be used (in spite of its notorious unreliability) the air-vent opening should be divided into eight equal areas, up to 10 in. square, and half-minute readings taken at the center of each area. When the eight areas exceeded the 10-in. limit, 12 equal areas were to be used. The average velocity was to be taken as the average reading—deducting in the process any negative readings from the total. For figuring volume the net area of the register was to be used except where the anemometer is held two inches away from the openings.

The presidential address of John R. Allen was read by Vice-President J. F. Hale, Prof. Allen being engaged in work at Constantinople. The address bore the title "Developments and Present Problems in Heating and Ventilation." It described the advance of present practice from a mechanics' art to an engineering science, and then led on to mention of the data which were available and those which were desirable but not in hand.

F. I. Cooper, of Boston, in the paper "Improved Air Conditions in a Residence," described the equipment placed in a Back Bay attic, which warmed, washed and ozonized the air which was fed to a few rooms. The work was attempted on the advice of physicians and was carried out with available only electric current and water under pressure.

The paper, "Downward Ventilation in a Schoolhouse," by C. E. Beery, of Rockford, Ill., described a heating and ventilating system where steam coils were suspended from the ceiling and warm fresh air was discharged from a duct head in the center of the room just below the plane

of the steam coils. The foul-air exits were in the corners at the floor. The author claimed that this design (patented) secured a temperature range of 1° F. throughout the room in the breathing zone, complete absence of drafts and cross-currents, improvement in work of teachers and pupils.

F. K. Chew, of New York, presented the paper, "An Analysis of a Combination Heating System," which was a description of hot-water and hot-air equipment in a converted farm house. The author put this material together that the society proceedings might show a complete record of all current methods of heating and ventilation.

The paper entitled "Design of Indirect Heating Systems With Respect to Maximum Economy and Operation," by F. L. Bucey and W. H. Carrier, of the Buffalo Forge Co., Buffalo, N. Y., presented a study of the economic balance between first cost and operating expense of a hot-blast heating system. They found that current practice resulted in much too high velocities through the coils and ducts causing too heavy an expenditure of power to result in a maximum economy over all. Starting with certain practical assumptions, they mathematically derived a basis for laying out a heating and ventilating system so as to obtain any desired economic results. They found that the most economical system was secured when the installation was proportioned so that the annual cost of power due to frictional resistance through the heater amounted to 28.6% of the annual interest and depreciation on the first cost of the heater regardless of variations in depth of heater, temperature rise, or steam pressure. They found the most economical velocity through ducts when the annual cost of power due to the pipe resistance was one-third the annual allowance for interest and depreciation, which was noted to be about 25% on the original cost of installation. The lower velocities brought about by these conditions required the use of larger fans to operate at higher efficiency. Considering the entire heating and ventilating system, the annual cost of power was held to be placed practically at 30% of the total annual allowance for interest and depreciation. Assuming that this allowance was 20%, as an average, they found 6% on the first cost as the most economical yearly rate ordinarily to be allowed for power.

In the discussion the point was raised as to the effect on this theory when the exhaust steam from the fan engine was used to heat the air blown through the coils. The authors admitted that in such cases the cost of engine power was often practically negligible, and that their conclusions would not hold then.

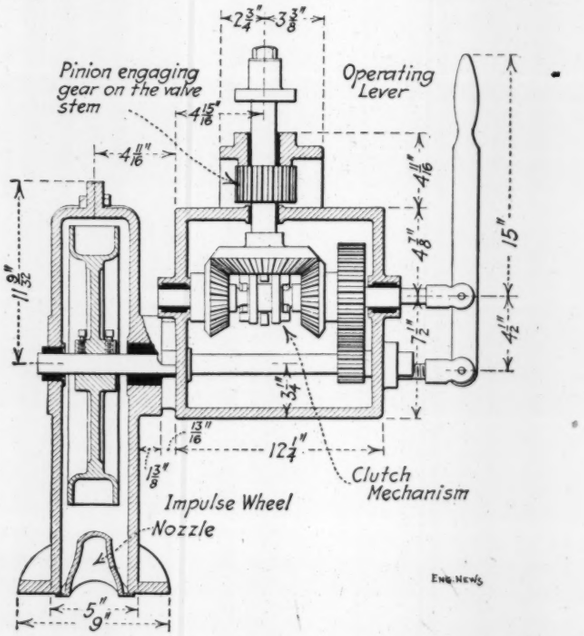
Under the title, "Ozone and Its Applications," M. W. Franklin, of Schenectady, N. Y., gave an informal presentation of facts in an interesting and entertaining way. The speaker did not claim that ozone was to supplant ventilation, but that there were certain instances where it was a very valuable adjunct, as where there were certain odors which ventilation did not satisfactorily remove, or only transferred from one vicinity to another. Use of ozonated air was shown to oxidize the odoriferous particles satisfactorily. A number of specific cases of work of this character were cited—glue factories, paint works, rendering plants, and rooms where offensive human emanations were encountered.

A New Valve-Operating Device

The Water-Works Department of the District of Columbia has recently adopted for use on all important valves throughout its system, a new hydraulic valve operator, manufactured by the Self-Operating Valve Co., of Washington, D. C.

One man can open or close the largest valve where this machine is attached, and the time required is from three to five minutes. Under the old system of manual operation, several men were required to open or close a large valve and the time required was from half an hour to an hour. Electric motors have been tried in this service and found an improvement on the manual operation, but in most cases the conditions of the service are such as to make electric motors subject to short-circuits and generally unreliable.

The hydraulic operator shown in the accompanying cut is simply a water motor driven by water from the main



SECTIONAL ELEVATION OF A NEW HYDRAULIC-VALVE OPERATOR, SHOWING REVERSING-CLUTCH MECHANISM
(Self Operating Valve Co., Washington, D. C., manufacturers)

and connected, through gearing, to the valve stem. The operating lever shown enables the motor to be stopped, started or reversed at will. By means of a clutch the device may be allowed to get up speed before it is thrown into mesh with the operating screw, if that is found desirable in cases where the gate is rusted or otherwise clogged. A limit-device is arranged to stop the water-supply to the motor when the gate has reached the end of its travel in opening or closing.

Among the advantages claimed for this device by the makers are its simplicity and durability, its speed of operation, and the fact that it takes up no more room than the valve alone, and will work equally well whether submerged or not.

The device is supplied either separate from the valve, in which case it may be easily and quickly attached to any form of gate valve already installed, or it can be obtained attached to any desired make of gate valve, supplying a complete self-operating valve ready for installation.

A Novel German Air Filter

By Otto E. Trautmann*

It is well known that with large and high-speed electric generators, air cooling is often resorted to in order to prevent injury from over-heating, and to keep down ratios of weight to output.

Since the air in the vicinity of a power plant is not particularly clean, much coal and metal dust is often blown into the machine windings, causing danger of short-circuits and making the cleaning of the machine a frequent necessity.

In Europe hardly a power plant is now erected without being equipped with air filters for the cooling air of the generators. These filters generally consist of strong cloth (specially woven for the purpose) which is stretched over frames of wood or iron. These frames are set in a zigzag fashion after the manner usually adopted in air-filter arrangement to allow sufficient surface for the passage of the air but, at best, are hard to keep clean and necessitate duplicate units or use of unfiltered air during cleaning.

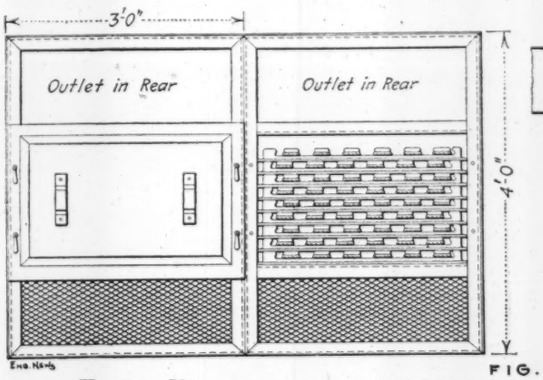


Fig. 1. Elevations of a 140,000-cu.ft. Air Filter

(Bollinger cotton-rope type)

The former power superintendent of the City of Charlottenburg, Germany, Mr. Bollinger, has designed a new type of filter whose advantages lie in compactness and ease of cleaning. This filter consists of single sections which are now manufactured in three standard sizes, each for 100,000, 140,000 and 175,000 cu.ft. of air per hr. These sections are built up to meet the capacity required and to suit the space available.

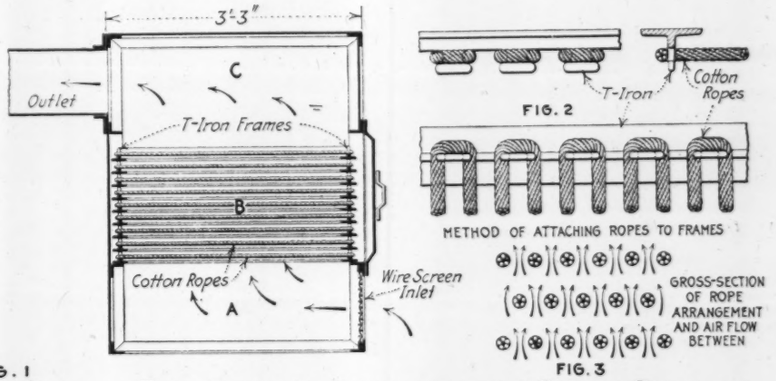
Fig. 1 shows the dimensions for two sections each of 140,000 cu.ft., and is fairly typical of other capacities, except as to size. The filtering elements are set in a housing of angles and sheet iron, comprising three compartments: (1) the air inlet, "A"; (2) the frame box, "B", and (3) the air outlet, "C." In the frame box are placed 20 to 25 frames, of small iron T-bars; each frame is equipped with cotton ropes strung very close to each other, giving but a small air passage between. Details of the method of attaching these ropes to the T-irons is shown in Fig. 2. These frames are put into position so that the spaces of the upper frame between the ropes are always vertically over the ropes of the next lower one, that is, staggered as shown in Fig. 3, forcing the air to change its direction of flow repeatedly while passing through.

*349 W. 57th St., New York City.

A slot for a sliding damper is provided so that it is possible to shut one section off at a time for cleaning. After shutting a section off by this damper the cover of the frame box is removed, when each rope frame can be withdrawn and cleaned.

It is generally the practice to use two sections as shown in Fig. 1, and a spare set of rope frames. Then one section has to do all the filtering during a minimum time only—the interval required in which to remove the clogged frames and to insert the clean ones in their place. This requires but a few minutes and throws extra load on the remaining frames for such a short time as to be practically negligible. The air inlet is covered with a wire-mesh screen for protection, while the air outlet or outlets are generally connected together into some sort of chamber from which ducts are run to desired points.

So far as the writer knows this system has been limited to use in power plants, and it is not positively known how successfully it could be applied in heating and ventilating work where air is filtered for breathing. However, the success attending its operation in Europe for power stations seems to show that in positions of limited space this device might also be made to answer the requirements of heating and ventilating installations.



Figs. 2 and 3. Attachment of Cotton Ropes in Bollinger Air Filter and Diagrammatic Cross-section of a Section

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The Complexity of Business done by the great German electrical manufacturing concern, the Allgemeine Elektricitäts Gesellschaft, is shown in a summary of the annual report printed in the "A. E. G. Journal" for December, 1912. Out of the business profits, a dividend of 14% will be distributed after setting aside 3,000,000 M. as a contribution to the reserve fund which now is increased to half the amount of the share capital.

The machines and transformers delivered from the Berlin factory numbered 118,205 and showed a horsepower output of 2,529,000. Among these machines were large three-phase generators for waterpower units up to 15,000 hp. and generators for coupling to Diesel motors up to 6000 hp. In the manufacture of steam turbines the speed limit was pushed up to 3000 r.p.m. for three-phase sets under full guarantee. There were 312 turbines put out having an output of 735,650 hp. The largest units constructed were for 20,000 hp.

The company also makes or directly controls concerns which make arc and incandescent lamps, arc-lamp carbons, mercury-vapor lamps, electric clocks, workmen's time registers, electrical instruments, tachometers for power-driven vehicles, typewriters, heating and cooking apparatus, porcelain fittings, ozone equipment, railway ticket-printing machines, electric wires and cables, enamel-insulated wire, pneumatic tires and hard- and soft-rubber articles, pleasure and commercial automobiles, motor boats and motors, airships and flying machines, rotary air compressors, etc. The company also participates in the formation of gas and electric-supply companies about Germany. The total number of employees amounts to 70,162.

The Oil Industry of California

The development of the oil resources and the oil industry of California is one of the remarkable features in the mineral industries of the United States. The oil available in the California field is from 30 to 50% of the total oil resources of the country according to the estimate of the U. S. Geological Survey, given in Table I.

TABLE I. OIL RESOURCES OF THE UNITED STATES.

Oil Fields	Million Barrels Minimum	Maximum
Appalachian	2,000	5,000
Lima-Indiana	1,000	3,000
Illinois	350	1,000
Mid-continent	400	1,000
Gulf	250	1,000
California	5,000	8,500
Minor fields	1,000	5,000
Total	10,000	24,500

As in the development of other natural resources, there has been lack of system and enormous waste, and although there is a movement to introduce improved methods, this is really only the commencement of work which should have been undertaken and carried into effect long ago. An interesting report on the oil situation in California was made some months ago to the Commercial Club of San Francisco by a committee on fuel resources,* and from this report we have compiled the accompanying article.

Oil Fields and Oil Production

The order of development of the present oil fields in California, and their production for the year 1911 are given in Table II, but most of the fields were known and experimented with for several years before the commercial development commenced. Table III shows the estimated area and production, from a table by F. A. Bell, of the U. S. Geological Survey, and Table IV shows the varying rate of daily production per well in some of the fields.

In regard to the total capacity of the oil fields, the report states that it is fashionable to talk of the unlimited oil resources of California, but that compared to the coal resources of Great Britain they are pitifully small. With an annual coal output of 25,000,000 tons and taking 3½ bbl. of oil equivalent to a ton of coal, the output of the latter would have to be 800 million barrels or ten times the present annual output. Ten years of such production would exhaust the oil resources of California, while Great Britain has coal resources estimated sufficient for 450 years at the present rate of production. Such a comparison is considered sufficient to justify the demand for the strictest conservation of California oil. The annual production of California oil (in thousands of barrels), has been approximately as follows:

1875	3	1888	690	1900	4,324
1880	41	1890	307	1905	33,427
1885	325	1895	1208	1910	73,070

Notes on Some of the Principal Oil Fields

Ventura—The first work in developing the oil industry was done by Josiah Stanford and his associates in Ventura County in 1864, their attention having been attracted by the seepage of oil from the exposed oil-sands at the foot of the southern slope of Sulphur Mountain. Tunnels 60 to 3000 ft. long have been driven into the face of the mountain to intercept the oil strata, the flow being by gravity to tanks near the mouths of the tunnels. Some of these are still producing oil in a small way. The first oil well in this field was drilled in 1866, but it was not until 1883 that the industry was fairly started. The oil is of superior quality for refining purposes, and at that time its use for fuel was not thought of. The production also was very limited, especially with the methods of drilling then in use. The development of modern well-drilling is a matter of the last decade, and is still progressing in an effort to meet problems that confront the industry.

In 1884, the necessity for a more extensive use of California oil was pressing upon the producers. With a view of creating an additional market the Union Oil Co. borrowed an old locomotive from the A., T. & S. F. Ry., and fitted it with oil-burning equipment. After months of experimental work, success was reached, and this opened a new era in the oil industry. It is said that this was the first oil-burning locomotive in the world, but we believe it had predecessors in Russia. In 1885 a 4-in. pipe line was laid from the Ventura County oil fields to the coast, and two steamers (each of 7500 bbl. capacity) were put in service between Ventura and Pacific coast points, especially San Francisco Bay.

Orange County—The Whittier-Fulton district did not become active until the value of fuel oil had been demonstrated, but it is now considered one of the best producing sections. Most of the product, however, is used locally. The Union Oil Co. has a pipe line of 15,000 bbl. daily capacity to tidewater at San Pedro, and the Standard Oil Co. has a 6-in. pipe line (20,000 bbl. capacity) to its new refinery at El Segundo, near Redondo Beach. As a whole, the fields in southern California are not regarded as of any such importance as those of the San Joaquin Valley.

Los Angeles—This is still an important factor and is one of the prime causes of the growth of Los Angeles, having given to that city a supply of fuel at its door.

Kern River—The area is about 12 sq.mi., and the large production did not begin until 1901-02. This field has been threatened seriously by water. The production has passed its maximum, and probably will diminish from year to year. An 8-in. pipe line to Point Richmond (on San Francisco Bay) was laid by the Standard Oil Co. in 1904-05; it is 280 miles long, and has a daily capacity of 15,000 to 20,000 bbl. The Associated Oil Co. now has a parallel 8-in. line to Port Costa, on San Francisco Bay.

Coalinga—Shallow wells were drilled at first, but in 1906-07 the Union Oil Co. drilled two wells 900 ft. and 1100 ft. deep, while in 1907 another firm struck 10 ft. of oil-sand at a depth of 300 to 400 ft.; this was the first strike of importance. Until the discovery of the Midway field, the Coalinga ranked as the most important. The Standard Oil Co. serves it by a branch pipe from Mendota, on its Kern River pipe line, and the Independent Oil Producers Agency has a pipe line south to Port Harford. The Associated Oil Co. has a branch from the Kern River line, and also a 6-in. pipe line from Coalinga to Monterey Bay.

McKittrick—The development of this field has progressed rapidly since 1900. The oil was transported by rail until 1908, when the Standard Oil Co. built an 8-in. pipe line to its main line at Kern. In 1909, the Producers Transportation Co. completed its pipe line to Port Harford and, in 1910, the Associated Oil Co. opened a line to this field.

*M. L. Requa, Chairman; F. W. Bradley, Mining Engineer; Walter Stadler, Geologist.

SUNSET—Various attempts have been made here since 1889, but nothing of importance was accomplished before 1902; the field had then a checkered career until 1908, when production increased rapidly.

MIDWAY—This was the sensation of the years 1910 and 1911, its production increasing from 2,234,530 bbl. in 1909 to 11,148,925 and 21,584,640 bbl. in the later years, respectively. The character of the oil is largely refining oil, but the water conditions are the most serious of any field, with the possible exception of the Kern field.

SANTA MARIA—This field, on the coast, was first developed systematically by the Western Union Oil Co. In 1904 the Hartnell gusher was opened by the Union Oil Co., and this was the largest producer in the history of the oil industry until the striking of the Lakeview gusher in the Midway field.

ESTIMATE OF RESOURCES AND PRODUCTION

The U. S. Geological Survey estimates 850 sq.mi. as the total oil-bearing area of California. Other estimates are 700 and 560 sq.mi. The oil-producing area is confined entirely to the district south of Coalinga, and for much of the territory north of that point geologists offer little hope of future production. Recent developments on Belridge and Lost Hills have in a measure closed the gap between Coalinga and McKittrick, and it is possible that oil may be discovered over a majority of the intervening territory. But much of this may be at depths too great for profitable production at present prices and, in fact, there are large areas fairly well known to be underlaid with oil which cannot be commercially profitable until the oil reaches a price of about $1 per bbl. at the well. Indications point to the area between Coalinga and Bakersfield as being by far the most important oil territory of the state.

Estimates of production are based upon the hypothesis that a foot of oil sand over one acre will produce a given number of barrels—generally assumed as approximately 500 bbl. per acre-foot of oil sand. This is a fairly conservative assumption, based upon the voids in sand, and is an estimate of the extraction of a small part of the contents. It has been proved beyond question that oil sands vary in saturation in the same well (when separated in some instances by but two or three feet of intervening impervious formation) from nothing to probably complete saturation. The factor of extraction assumed, however, is so small that it allows for such variation, and while we may have 100 ft. of oil sand, of which but 25 ft. may be productive, the saturation of that 25 ft. may be so great as to give a production equivalent to 500 bbl. per acre-foot over the entire 100 ft. This is the first great assumption in calculating the probable production of California oil fields.

To maintain the present production of 78,000,000 bbl. per annum, it will probably be necessary to bring in annually new wells of a capacity of 15,000,000 or 16,000,000 bbl. at least. This is based upon a 20% decrease in the annual production, which must be supplied by new wells to take the place of the decreased production of old wells. The figure is an arbitrary one, probably not exact, but it is believed to be fairly within reason. If the average new well produced 40,000 bbl. per annum, or 150 bbl. per day, it will take approximately 400 wells drilled annually to maintain present production. The figure given of 150 bbl. per day is an extremely liberal one.

If we assume the average cost per well at $20,000, it will require annually the investment of $8,000,000 to drill the 400 wells necessary to maintain production; or, stating it another way, a charge of 11c. per bbl. of oil produced will be necessary to offset decline due to diminished production per well. If we assume $40,000 as the cost per well, a figure that will ultimately be reached, this charge becomes 22c. based on present production.

COST OF PRODUCTION

It is becoming more and more apparent that past estimates on the part of oil producers of California as to the cost of production have not been sufficiently high. Interest on capital, amortization of capital, cost per barrel for drilling to maintain production, depreciation of surface equipment, have in many cases not entered into the calculations in figuring cost of production. The figures herewith for a property of a depth of approximately 2500 ft. will give a fair idea of actual cost of operation:

	Cost per bbl
Pumping	$0.032
Pulling	0.0155
Bailing and cleaning	0.0126
Maintenance and repairs, including redrilling	0.10
Cost of drilling to maintain production	0.15
Amortization fund	0.08
Total cost per barrel	$0.3901

These cost figures vary and are less for properties with shallow wells, and also less for properties such as some in the Midway field, where the production per well is large. Taking the state as a whole, however, and for territory of this depth, the figures are believed to be fairly representative. In addition to this, there are unquestionably large areas in California where the cost of production, due to various causes (principally to increased depth or heavy-gravity oil) will cause production to be more expensive than herewith indicated. There is undoubtedly territory that will, at some future time, produce oil where the actual cost of production will approximate 50c. to 75c., and possibly in excess of $1 a barrel. In order that satisfactory profit may be realized, and by satisfactory profit is meant a profit commensurate with the hazards of the business, it is quite evident that prices for oil must be materially in excess of those now obtaining.

CONSERVATION OF THE OIL RESOURCES

The report mentioned states that the factors menacing the California oil resources may be summarized as follows, in the order of their importance: 1, influx of water to the oil formation; 2, competitive drilling; 3, unsatisfactory control of production, permitting an unnecessarily large production, with consequent loss through evaporation and seepage; 4, undetermined disposition of government lands.

1. WATER—Within the past few years the entrance of water into the oil fields has assumed a somewhat alarming aspect in some of the fields, and if the life of any field is cut short it will be due probably to this cause rather than to exhaustion of the oil. This water (upon which the oil may have been floating) may be an indication of the exhaustion of the oil, or it may come from other causes. Whether water necessarily accompanies oil is a debatable question, but it occurs in all the California fields, and the cementing of the well to exclude water is a difficult problem. In most cases the water can be controlled in this way, but it is not being done effectively. The remedy proposed in the report is to levy a special tax in each field, the money to be expended in that field under the direction of the State Mining Bureau in properly shutting off the water. All estimates of life and production of the oil fields may be nullified by the inflow of water, although some operators consider that the oil is simply driven to other parts of the field and is eventually recovered from wells at its edge.

Top water overlies the oil sands, and can be excluded by proper cementing of the wells. Bottom water underlies the oil sands, and if it is in the same stratum as the sands (and has the oil floating upon it) it cannot be prevented, as the exhaustion of the oil permits the water to rise and gradually drown first the deeper wells and then the shallower wells, until the field is flooded. If it is in strata underlying the oil sands, it can be kept out by not drilling into these strata, or by promptly and effectively plugging wells which do penetrate them. Intermediate water is inclosed in the formation and lies between an upper and lower stratum of oil-bearing sand. Trouble

TABLE II. OIL FIELDS OF CALIFORNIA

Field	Date of Development	Production in 1911 (1000 bbl.)
Ventura County (coast)	1864	459
Orange County (Whittier dist.)	late '70s	1,154
Los Angeles City (coast)	1892	437
Coalinga (valley)	1897	17,830
Kern River (valley)	1899	12,161
Salt Lake (coast)	1902	2,795
McKittrick (valley)	1900	5,373
Santa Maria (coast)	1901	6,228
Sunset (valley)	1903	4,626
Midway (valley)	1909	20,967

TABLE III. ESTIMATED AREA, PRODUCTION AND FUTURE SUPPLY OF CALIFORNIA OIL FIELDS.

Fields	Area acres	Production to Jan. 1, '12 million bbl.	Remaining supply million bbl.
Coalinga	30,000	98	1027
Lost Hills and Belridge	4,000	....	150
McKittrick	2,400	30	100
Midway-Sunset	40,000	60	1440
Kern River	6,000	144	446
Santa Barbara	6,800	40	131
Whittier (Orange Co.) and all east of Los Angeles	5,000	39	411
Salt Lake and all west of Los Angeles	2,000	38	34
Miscellaneous (incl. Ventura)	3,000	13	54
Total	99,200	462	3793

TABLE IV. VARIATIONS IN FLOW OF OIL WELLS IN CALIFORNIA

(Bbl. per well per day).

Oil Fields	1903	1905	1907	1909	1911
Kern River	82	63	40	32	24
Coalinga	72	124	99	86	68
McKittrick	73	79	102	116	84
Sunset	44	28	47	48	78
Midway	....	....	....	83	132
Santa Maria	('04) 91	291	291	171	119

from it can be minimized by drilling only the upper stratum first, and after its exhaustion, drilling the lower stratum, allowing the intermediate water to flood the upper one and possibly carry the unextracted oil to a higher level so that it may be recovered. Edge water is that on the edge of the oil field, along the strike of the formation, and its inflow is not preventable.

2. COMPETITIVE DRILLING—The existing method of drilling wells along the property lines makes it hopeless to secure any united curtailment of production, since adjacent companies are forced to continue production, for if either one stopped, the wells of the other would draw oil from the former's property. The report states that line drilling is done to get oil from adjacent property. If one company drills in a corner of four adjoining properties, each of the other three must drill a well or lose a certain quantity of its oil. The remedy suggested is to induce or compel the companies to drill from the center of the property outward. This would be economical in keeping the wells as near as possible to the power plant, and when the price of oil is low the production could be reduced without fear of having the property drained by the wells of an adjacent company.

3. CONTROL OF PRODUCTION—The marketing of oil, and especially fuel oil, must usually be done by a few large companies, according to the report. Under the practice of contracting the oil for a period of years it is almost impossible for small concerns to enter the marketing field, and consumers will not jeopardize their business by contracting with these small concerns. The business necessitates a large expenditure for pipe lines, distributing stations and ships, so that it is impossible for the small producer to market his oil himself. The law does not permit of pooling agreements, but the report suggests that the large marketing companies should be permitted to make agreements and price regulations (subject to government supervision), so that production may be controlled.

4. DISPOSITION OF GOVERNMENT LAND—The United States government has in California a large amount of oil-bearing land that is withdrawn from entry, and this is a desirable situation since there is no present market for the oil in this teritory. The method of disposing of the lands has yet to be worked out, but indications point to a leasing system. The report states that this should be as free from government control as possible, although the government might exercise police powers as to shutting off water from the wells and also require a standard system of accounting. It would be very unwise to place all of this land on the market at one time.

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Swimming Pool in the New Gymnasium of the Rensselaer Polytechnic Institute, Troy, N. Y.*

The Class of '87 of the Rensselaer Polytechnic Institute gave, last year, $150,000 to the Trustees of the Institute for the construction of a new gymnasium. This has just been finished. It contains rooms for general gymnastic exercise, basketball, indoor baseball practice, wrestling and fencing, a squash court, four bowling alleys, shower baths, and a room with 800 lockers. It contains also a carefully designed and constructed swimming pool. As such pools quite often leak, sometimes to such an extent that they have to be relined and made smaller, a description of this one, which represents the best modern practice, should be of interest.

The pool is placed in the basement of the gymnasium in a room 97 ft. long, 40 ft. wide, and 18 ft. 6 in. high. There is a gallery, hung from the ceiling, around the four sides of the room. The side walls are covered with white tile, 3x6 in. in size, from the floor to the under side of the white marble slabs which form the floor of the gallery. The floor around the pool is of terrazzo. The room is well lighted by windows opening on two sides on the gallery floor level; it is also lighted by electricity.

The pool is 75 ft. long and 30 ft. wide with a depth of 8 ft. at one end and 4 ft. at the other. It is tiled on the bottom and sides including the scum gutters, with ¾ in. square white ceramic tile laid in cement.

The pool was constructed as follows: A bed of clean ashes, 12 in. thick, was placed on the clay foundation. On this was placed a layer of concrete 4 in. thick. Above this came a waterproofing course consisting of a heavy liquid asphalt into which, while hot, a layer of tarred felt, weighing 15 lb. per 100 sq.ft., was imbedded. This was followed by alternating layers of felt and asphalt until five layers of felt and six moppings of asphalt were applied. All the felt was bedded into the asphalt while the latter was hot, and each layer of felt was completely covered by asphalt. A bed of concrete, 12 in. thick, reinforced with galvanized wire cloth, was laid on the felt. The wire was No. 3 of 3x3-in. mesh and the sheets of wire were turned over and locked to each other over the entire bed of the pool. Near the sides of this 12 in. bed of concrete twisted steel bars 10 ft. long of ½ in. square cross section, spaced 12 in. c. to c., were imbedded. Seven feet of the length of these bars was imbedded in the 12-in. course and the bars were turned up so that 3 ft. of their length was in concrete side walls, which were carried up from the 12-in. bed. This was done all around the pool

*From information furnished by the architects.

and at the four corners these bars were curved in basket form. The bars hold the bed and side walls together and materially help to form a tight pool. The top of this bed and the side walls were then covered with heavy liquid asphalt and five layers of felt and moppings of asphalt were applied as described above for the waterproofing beneath. An additional layer of felt and asphalt was applied at all angles of the pool.

The sides of the pool were then lined with a brick wall 8 in. thick and the bottom was covered with a layer of concrete 4 in. thick which closely fitted against the wall. All interstices between the brick wall and the waterproofing behind it were filled with grout. This is important. No interstices should be left to exert pressure behind the lining of the pool when the water is drawn off. A pool constructed in New York some years ago had the slabs of marble lining it thrown out by the water pressure behind them when the water was let out of the pool. Water in interstices behind the lining cannot escape as quickly as the water in the pool and pressure behind the lining under considerable head may result.

The bottom and sides of the pool were then lined with ¾-in. square white ceramic tile laid in cement mortar.

The scum gutter, shown in section in the figure, extends all the way around the pool. It carries off any scum and also any excess of water in the pool. It is drained by twelve 3-in. pipes at equal intervals around the pool. The ends of these pipes are covered with strainers in the bottom of the gutter. There is also a 4-in. overflow pipe at one end of the pool, with a strainer placed about ½ in. above the bottom of the gutter. Cleanouts are provided for all these overflows.

Provision is made for draining any water which may percolate through the upper layer of waterproofing under the bottom of the pool by means of a 4-in. pipe which leads to a waterproofed manhole outside the pool in the machinery room. A float is provided to indicate the presence of water in this manhole. Such water may then be pumped out. The object is to prevent upward water pressure under the bottom when the pool is empty.

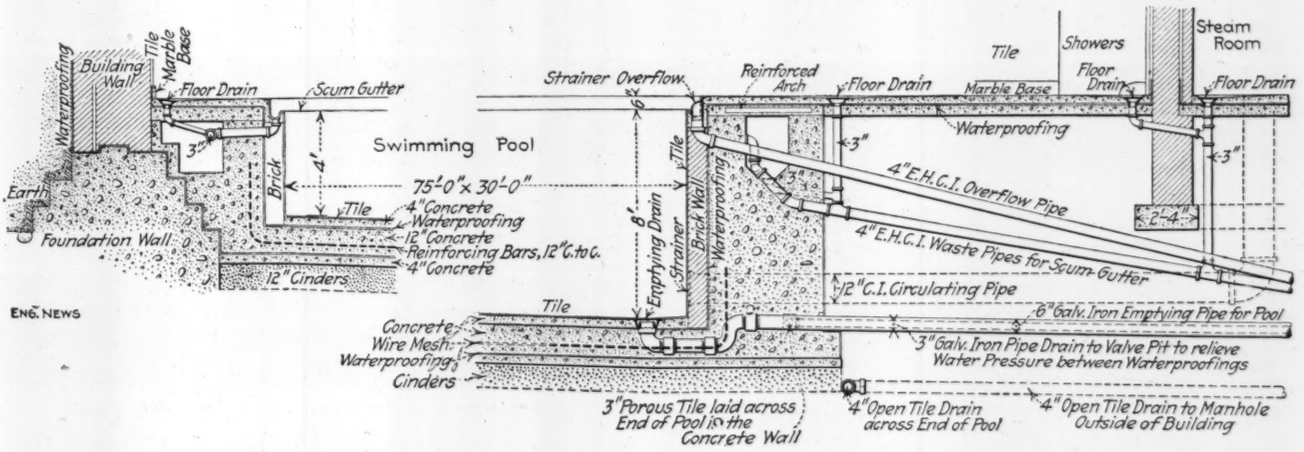
The water is filtered by mechanical filters before it enters the pool. It passes from a heater, where the temperature is automatically kept constant to the filters and thence to the pool. Two inch brass pipes carry it from the heater to two points of discharge; one on each side of the pool near the shallow end. A 12-in. cast-iron circulating pipe extends from the deep end of the pool near the bottom to the machinery room. This acts as a sump for an electric pump which forces the water through the heater and filters back to the pool.

Any water on the terrazzo floor surrounding the pool is drained away from the scum gutters to strainers connected with pipes which carry it to the scum gutter drainage system independently of the scum gutters. This keeps any dirty water on the floor from being swept into the pool.

Eight shower baths and a steam room are placed at one end of the room containing the pool.

The pool has been kept full of water for about a month and no leakage has resulted.

Messrs. Lawlor & Haase, 69 Wall St., New York City, are the architects of the building. The steam heating and filter systems were designed by Prof. Arthur M. Greene, and the electrical equipment by Dr. W. L. Robb, both of the Institute faculty. Dr. D. A. Sargent of Harvard University was consulted during the preparation of the plans, and additions to the equipment have been made by Mr. P. B. Samson, who was appointed Professor of Physical Training during the construction.



DETAILS OF CONSTRUCTION OF SWIMMING POOL, RENSSELAER POLYTECHNIC INSTITUTE, TROY, N. Y.

❖

Gas-pressure Regulations have been fixed for the Boroughs of Brooklyn and Queens, New York City, by the Public Service Commission for the First District of the state. One ward embracing Coney Island, the famous amusement resort, is not covered, action being postponed here until the summer conditions can be studied. The orders adopted here are similar in form and substance to those adopted for Boroughs of Manhattan, Bronx and Richmond, after a study of the gas-supply service there (see "Engineering News," Oct. 31, 1912).

It is provided that the companies shall install by Aug. 1, 1913, recording pressure gages so located that no consumer shall be more than 3800 ft. distant from one, and that there shall be at least one in each square mile. After Dec 1, 1913, the companies must maintain a minimum pressure of 2-in. water column on two consecutive days except in certain wards, and except where the service comes from a main located previous to the date of the order in an ungraded street or in a street not opened by the city, in which case a minimum pressure of 1½ in. is allowed until Dec. 1, 1915. The maximum pressure for this territory is 6 in. on two consecutive days. The maximum daily pressure variations are fixed for after Dec. 1, 1913, at 3-in. water column on two consecutive days, 2½ in. after Dec. 1, 1914, and 2 in. after Dec. 1, 1915. The maximum momentary pressure variation must not exceed 0.8 in. after Sept. 1, 1915, and the maximum pulsating pressure variation for after Nov. 1, 1913, is fixed at the same value, and reduced to 0.5 in. for after May 1, 1914.

Joint Committee on Concrete and Reinforced Concrete; Abstract of Second Report

On June 17, 1904, delegates or special committees on concrete from four technical societies met and formed the Joint Committee on Concrete and Reinforced Concrete, with the object of studying and reporting upon the making and use of concrete, especially reinforced concrete. The societies thus coöperating were: the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance-of-Way Association, and Association of American Portland Cement Manufacturers. In 1909 the Joint Committee rendered a report. Critical discussion of this report followed. For the past two years the Joint Committee has been considering changes in the report, and on Nov. 20, 1912, the revised or second report was adopted by the committee. The first public presentation of the new report was made to the American Society of Civil Engineers on Jan. 15, 1913.

The report comprises:

a. An account of the Joint Committee's work.
b. A historical review of cement and concrete.
c. Technical discussion of concrete and its use.
d. Recommended working stresses.
e. The cement specifications of the American Society for Testing Materials.
f. The steel specifications of the American Railway Engineering Association.
g. Formulas for designing reinforced-concrete beams and columns, based on straight-line stress-distribution, and identical with what all modern textbooks on the subject give.

Below we reproduce about one-third of the full report; we omit a, b, e and f, and give extracts only of c; d is given nearly in full, and g in full.

In reporting the meeting of the American Society of Civil Engineers (p. 182, issue of Jan. 23, 1913) we enumerated the chief points in which the second report of the Joint Committee differs from the first report. The changes are numerous, but none of them appear to be radical.

Since the appearance of the first Progress Report, in 1909, many experiments have been conducted by some of the technical institutions and by private and corporate interests, and through these and through longer experience in construction by its members and others, the Committee is now able to make some perfecting modifications of its former report and to add some entirely new material.

While the report deals with every kind of stress to which concrete is subjected, and includes all ordinary conditions of proportioning and handling, it does not go into all types of construction or all the applications to which concrete and reinforced concrete may be put.

This report is what the word implies, and nothing more; it is not a "specification," but may be used as a basis for specifications.

RESPONSIBILITY AND SUPERVISION

The design of reinforced-concrete structures should receive at least the same careful consideration as those of steel, and only engineers with sufficient experience and good judgment should be intrusted with such work.

The computations should include all minor details, which are sometimes of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connections between the component parts, so that they cannot be displaced. As the connections between reinforced-concrete members are frequently a source of weakness, the design should include a detailed study of such connections, accompanied by computations to prove their strength.

The construction of buildings in large cities is regulated by ordinances or building laws, and the work is inspected by municipal authorities. For reinforced-concrete work, however, the limited supervision which municipal inspectors are able to give is not sufficient. Therefore, means for more adequate supervision and inspection should be provided. The engineer who prepares the design and specifications should have the supervision of the execution of the work.

The Committee recommends the following rules for structures of reinforced concrete for the purpose of fixing the responsibility and providing for adequate supervision during construction:

(a) Before work is commenced, complete plans shall be prepared, accompanied by specifications, stress computations and descriptions showing the general arrangement and all details. The plans shall show the size, length, dimensions for points of bending, and exact position of all reinforcement, including stirrups, ties, hooping and splicing. The computations shall give the loads assumed separately, such as dead and live loads, wind and impact, if any, and the resulting stresses.

(b) The specifications shall state the qualities of the materials to be used for making the concrete, and the manner in which they are to be proportioned.

(c) The strength which the concrete is expected to attain after a definite period shall be stated in the specifications.

(d) The drawings and specifications shall be signed by the engineer and the contractor.

(e) Plans and specifications for all public structures should be approved by a legally authorized State or City official, and copies of such plans and specifications placed on file in his office.

(f) The approval of plans and specifications by other authorities shall not relieve the engineer or the contractor of responsibility.

(g) Inspection during construction shall be made by competent inspectors employed by and under the supervision of the engineer, and shall cover the following: 1. The materials. 2. The correct construction and erection of the forms and the supports. 3. The sizes, shapes and arrangement of the reinforcement. 4. The proportioning, mixing and placing of the concrete. 5. The strength of the concrete, by tests of standard testpieces made on the work. 6. Whether the concrete is sufficiently hardened before the forms and supports are removed. 7. Prevention of injury to any part of the structure by and after the removal of the forms. 8. Comparison of dimensions of all parts of the finished structure with the plans.

(h) Load tests on portions of the finished structure shall be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired. Loading shall be carried to such a point that one and three-quarters times the calculated working stresses in critical parts are reached, and such loads shall cause no injurious permanent deformations. Load tests shall not be made until after 60 days of hardening.

DESTRUCTIVE AGENCIES

(a) **Corrosion of Metal Reinforcement**—Tests and experience indicate that steel sufficiently embedded in good concrete is well protected against corrosion, no matter whether located above or below water level. It is recommended that such protection be not less than 1 in. in thickness. If the concrete is porous, so as to be readily permeable by water, as when the concrete is laid with a very dry consistency, the metal may corrode on account of the presence of moisture and air.

(b) **Electrolysis**—The most recent experimental data available on this subject seem to show that while reinforced-concrete structures may, under certain conditions, be injured by the flow of electric current in either direction between the reinforcing material and the concrete, such injury is generally to be expected only where voltages are considerably higher than those which usually occur in concrete structures in practice. If the iron be positive, trouble may manifest itself by corrosion of the iron accompanied by cracking of the concrete, and, if the iron be negative, there may be a softening of the concrete near the surface of the iron, resulting in a destruction of the bond. The former or anode effect decreases much more rapidly than the voltage, and almost if not quite disappears at voltages that are most likely to be encountered in practice. The cathode effect, on the other hand, takes place even on very low voltages, and is therefore more important from a practical standpoint than that of the anode.

Structures containing salt or calcium chloride, even in very small quantitites, are very much more susceptible to the effects of electric currents than normal concrete, both the anode and cathode effects progressing much more rapidly in the presence of chlorine.

There is great weight of evidence to show that normal reinforced-concrete structures free from salt are in very little danger under most practical conditions, while non-reinforced concrete structures are practically immune from electrolysis troubles.

The results of experiments now in progress may yield more conclusive information on this subject

(c) **Sea Water**—The data available concerning the effect of sea water on concrete or reinforced concrete are limited and inconclusive. To effect the best resistance to sea water, the concrete must be proportioned, mixed and placed so as to prevent the penetration of sea water into the mass or through the joints. The concrete should be kept from exposure to sea water until it is thoroughly hard and impervious.

(d) **Acids**—Concrete of first-class quality, thoroughly hardened, is affected appreciably only by strong acids which seriously injure other materials.

(e) **Oils**—When concrete is properly made and the surface is carefully finished and hardened, it resists the action of such mineral oils as petroleum and ordinary engine oils. Oils which contain fatty acids produce injurious effects, forming compounds with the lime which result in a disintegration of the concrete in contact with them.

(f) **Alkalies**—The action of alkalies on concrete is problematical. In the reclamation of arid land, where the soil is heavily charged with alkaline salts, it has been found that concrete, stone, brick, iron and other materials are injured under certain conditions. Such destructive action can be prevented by the use of a protective coating, and is mimimized by securing a dense concrete.

MATERIALS

Cement—Only portland cement is suitable for reinforced concrete. The cement should meet the requirements of the Standard Methods of Testing and the Specifications for Cement, the result of the joint labors of special committees of the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering Association, and others.

Aggregates—Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density* or a minimum percentage of voids.

(a) Fine Aggregate—This should consist of sand, crushed stone, or gravel screenings, graded from fine to coarse, and passing when dry a screen having holes ¼ in. in diameter; it is preferable that it be of silicious material, and should be clean, coarse, free from dust, soft particles, vegetable loam or other deleterious matter; and not more than 6% should pass a sieve having 100 meshes per lin.in. Fine aggregates should always be tested.

Fine aggregate should be of such quality that mortar composed of one part portland cement and three parts fine aggregate by weight, when made into briquettes, will show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.† If the aggregate be of poorer quality, the proportion of cement in the mortar should be increased to secure the desired strength.

If the strength developed by the aggregate in the 1:3 mortar is less than 70% of the strength of the Ottawa sand mortar, the material should be rejected. To avoid the removal of any coating on the grains, which may affect the strength, bank sands should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture may be determined on a separate sample for correcting weight. From 10 to 40% more water may be required in mixing bank or artificial sands than for standard Ottawa sand to produce the same consistency.

(b) Coarse Aggregate—This should consist of crushed stone or gravel which is retained on a screen having holes ¼ in. in diameter, and graded from the smallest to the largest particles; it should be clean, hard, durable and free from all deleterious matter. Aggregates containing dust and soft, flat or elongated particles, should be excluded from important structures..

The maximum size of the coarse aggregate is governed by the character of the construction.

For reinforced concrete and for small masses of unreinforced concrete, the aggregate must be small enough to produce with the mortar a homogeneous concrete of viscous consistency which will pass readily between and easily surround the reinforcement and fill all parts of the forms.

For concrete in large masses, the size of the coarse aggregate may be increased, as a large aggregate produces a stronger concrete than a fine one, although it should be noted that the danger of separation from the mortar becomes greater as the size of the coarse aggregate increases.

Cinder concrete should not be used for reinforced-concrete structures. It may be allowable in mass for very light loads or for fire protection purposes. The cinders used should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal or ashes.

Metal Reinforcement—The Committee recommends, as a suitable material for reinforcement, steel filling the requirements for structural steel reinforcement of the specifications adopted by the American Railway Engineering Association. Where little bending or shaping is required, and also for reinforcement for shrinkage and temperature stresses, material filling the requirements of the specifications adopted by the American Railway Engineering Association for high-carbon steel may be used, adopting the same unit-stress as hereinafter recommended for structural-grade material.

For the reinforcement of slabs, small beams or minor details, or for reinforcement for shrinkage and temperature stresses, wire drawn from bars of the grade of rivet steel may be used, with the unit-stresses hereinafter recommended.

The reinforcement should be free from excessive rust, scale or coatings of any character which would tend to reduce or destroy the bond.

PREPARING AND PLACING

Proportions—For reinforced-concrete construction, one part of cement to a total of six parts of fine and coarse aggregates measured separately should generally be used. For columns, richer mixtures are generally preferable, and in massive masonry or rubble concrete, a mixture of 1:9 or even 1:12 may be used. These proportions should be determined by the strength or the wearing qualities required in the construction at the critical period of its use.

Removal of Forms—So many conditions affect the hardening of concrete that the proper time for the removal of the forms should be decided by some competent and responsible person, especially where the atmospheric conditions are unfavorable.

It may be stated in a general way that forms should remain in place longer for reinforced concrete than for plain or massive concrete, and that forms for floors, beams and similar horizontal structures should remain in place much longer than for vertical walls.

When the concrete gives a distinctive ring under the blow of a hammer, it is generally an indication that it has hardened sufficiently to permit the removal of the forms with safety. If, however, the temperature is such that there is any possibility that the concrete is frozen, this test is not a safe reliance, as frozen concrete may appear to be very hard.

DETAILS

Shrinkage and Temperature Changes—Shrinkage of concrete, due to hardening and contraction from temperature changes, causes cracks the size of which depends on the extent of the mass. The resulting stresses are important in monolithic construction, and should be considered carefully by the designer; they cannot be counteracted successfully, but the effects can be minimized.

Large cracks, produced by quick hardening or wide ranges of temperature, can be broken up to some extent into small cracks by placing reinforcement in the concrete; in long continuous lengths of concrete, it is better to provide shrinkage joints at points in the structure where they will do little or no harm. Reinforcement is of assistance, and permits longer distances between shrinkage joints than when no reinforcement is used.

Small masses or thin bodies of concrete should not be joined to larger or thicker masses without providing for shrinkage at such points. Fillets similar to those used in metal castings, but of larger dimensions, for gradually reducing from the thicker to the thinner body, are of advantage.

Shrinkage cracks are likely to occur at points where fresh concrete is joined to that which is set, and hence, in placing the concrete, construction joints should be made on horizontal and vertical lines, and, if possible, at points where joints would naturally occur in dimension-stone masonry.

Fireproofing—Experience, together with the results of tests thus far made, indicates that concrete, on account of its low rate of heat conductivity and the fact that it is incombustible, may be used safely for fireproofing purposes. The dehydration of concrete probably begins at about 500° F., and is completed at about 900° F., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehydration is very much re-

*A convenient coefficient of density is the ratio of the sum of the volumes of materials contained in a unit volume to the total unit volume.

†A natural sand obtained at Ottawa, Ill., passing a screen having 20 meshes and retained on a screen having 30 meshes per lin.in.; prepared and furnished by the Ottawa Silica Company, for 2 cents per lb., f.o.b. cars, Ottawa, Ill., under the direction of the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers.

tarded. The concrete that is actually affected by fire remains in position and affords protection to the concrete beneath it. For ordinary conditions, it is recommended that the metal in girders and columns be protected by a minimum of 2 in. of concrete; that the metal in beams be protected by a minimum of 1½ in. of concrete; and that the metal in floor slabs be protected by a minimum of 1 in. of concrete.

It is recommended that, in monolithic concrete columns, the concrete to a depth of 1½ in. be considered as protective covering, and not included in the effective section.

It is recommended that the corners of columns, girders and beams be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one.

DESIGN OF MASSIVE CONCRETE

In the design of massive or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be proportioned so as to avoid tensile stresses, except in slight amounts to resist indirect stresses.

Massive concrete is suitable for dams, retaining walls, and piers and short columns in which the ratio of length to least width is relatively small. Under ordinary conditions, this ratio should not exceed six. It is also suitable for arches of moderate span, where the conditions as to foundations are favorable.

DESIGN OF REINFORCED CONCRETE

Lengths of Beams and Columns—The span length for beams and slabs shall be taken as the distance from center to center of supports, but need not be taken to exceed the clear span plus the depth of beam or slab. Brackets shall not be considered as reducing the clear span in the sense here intended.

The length of columns shall be taken as the maximum unsupported length.

Internal Stresses—As a basis for calculations relating to the strength of structures, the following assumptions are recommended:

1. Calculations will be made with reference to working stresses and safe loads, rather than with reference to ultimate strength and ultimate loads.
2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression, within the usual limits of working stresses, is constant. The distribution of compressive stresses in beams, therefore, is rectilinear.
4. In calculating the moment of resistance of beams, the tensile stresses in the concrete are neglected.
5. Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses, the two materials, therefore, are stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken at 15, except as modified in Chapter VIII, Section 8.
7. Initial stress in the reinforcement, due to contraction or expansion in the concrete, is neglected.

It is recognized that some of the assumptions given herein are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

Deflection of Beams—This is affected by the tensile strength developed throughout the length of the beam. For calculations of deflections, a value of 8 for the ratio of the moduli will give results corresponding approximately with the actual conditions.

T-Beams—In beam-and-slab construction, an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used, extending over the beam and well into the slab.

Where adequate bond and shearing resistance between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall be determined by the following rules: (a) It shall not exceed one-fourth of the span length of the beam; (b) Its overhanging width on either side of the web, shall not exceed four times the thickness of the slab.

In the design of T-beams acting as continuous beams, due consideration should be given to the compressive stresses at the support.

Beams in which the tee form is used only for the purpose of providing additional compression area of concrete should preferably have a width of flange not more than three times the width of the stem and a thickness of flange not less than one-third of the depth of the beam. Both in this form and in the beam and slab form, the web stresses and the limitations in placing and spacing the longitudinal reinforcement will probably be controlling factors in design.

Floor Slabs—Floor slabs should be designed and reinforced as continuous over the supports. If the length of the slab exceeds one and five-tenth times its width, the entire load should be carried by transverse reinforcement. Square slabs may well be reinforced in both directions.*

The continuous flat slab with multiple-way reinforcement is a type of construction used quite extensively, and has recognized advantages for special conditions, as in the case of warehouses with large, open, floor space. At present, a considerable difference of opinion exists among engineers as to the formulas and constants which should be used, but experience and tests are accumulating data which it is hoped will in the near future permit the formulation of the principles of design for this form of construction.

The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beam, and its distribution will depend on the relative stiffness of the slab and the supporting beam. The distribution under ordinary conditions of construction may be expected to be that in which the load on the beam varies in accordance with the ordinates of a parabola having its vertex at the middle of the span. For any given design, the probable distribution should be ascertained, and the moments in the beam calculated accordingly.

Continuous Beams and Slabs—When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment, and the stresses in concrete recommended farther below should not be exceeded. In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

(a) That for floor slabs, the bending moments at center and at support be taken at $\frac{wl^2}{12}$ for both dead- and live-loads, where w represents the load per linear foot and l the span length.

(b) That for beams, the bending moment at center and at support for interior spans be taken at $\frac{wl^2}{12}$, and for end spans it be taken at $\frac{wl^2}{10}$ for center and adjoining support, for both dead- and live-loads.

(c) In the case of beams and slabs continuous for two spans only, the bending moment at the central support should be taken as $\frac{wl^2}{8}$ and near the middle of the span as $\frac{wl^2}{10}$.

(d) At the ends of continuous beams, the amount of negative moment which will be developed will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. There will usually be some restraint, and there is likely to be considerable. Provision should be made for the negative bending moment, but, as its amount will depend on the form of construction, the coefficient cannot be specified here, and must be left to the judgment of the designer.

For spans of unusual length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Even if the center of the span is designed for a greater bending moment than is called for by a or b, the negative moment at the support should not be taken as less than the values there given.

Bond Strength, and Spacing of Reinforcement—Adequate bond strength should be provided. The formula hereinafter given for bond stresses in beams is for straight longitudinal bars. In beams in which a portion of the rein-

*The exact distribution of load on square and rectangular slabs, supported on four sides and reinforced in both directions, cannot readily be determined. The following method of calculation is recognized as faulty, but it is offered as a tentative method which will give results on the safe side. The distribution of load is first to be determined by the formula:

$$r = \frac{l^4}{l^4 + b^4}$$

in which r = proportion of load carried by the transverse reinforcement, l = length, and b = breadth of slab. For various ratios of $\frac{l}{b}$ the values of r are as follows:

$\frac{l}{b}$	r
1	0.50
1.1	0.59
1.2	0.67
1.3	0.75
1.4	0.80
1.5	0.83

Using the values above specified, each set of reinforcement is to be calculated in the same manner as slabs having supports on two sides only, but the total amount of reinforcement thus determined may be reduced 25%, by gradually increasing the rod spacing from the third-point to the edge of the slab.

forcement is bent up near the end, the bond stress at places in both the straight bars and the bent bars will be considerably greater than for all the bars straight, and the stress at some point may be several times as much as that found by considering the stress to be uniformly distributed along the bar. In restrained and cantilever beams, full tensile stress exists in the reinforcing bars at the point of support, and the bars must be anchored in the support sufficiently to develop this stress.

In case of anchorage of bars, an additional length of bar must be provided beyond that found on the assumption of uniform bond stress, for the reason that, before the bond resistance at the end of the bar can be developed, the bar may have begun to slip at another point, and "running" resistance is less than the resistance before slip begins.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength; but it should be recognized that, even with a deformed bar, initial slip occurs at early loads, and that the ultimate loads obtained in the usual tests for bond resistance may be misleading. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but, as an additional safeguard, such anchorage may properly be used in special cases. Anchorage furnished by short bends at a right angle is less effective than hooks consisting of turns through 180°.

The lateral spacing of parallel bars should not be less than three diameters, from center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than 1 in. The use of more than two layers is to be discouraged, unless the layers are tied together by adequate metal connections, particularly at and near points where bars are bent up or bent down.

Diagonal Tension and Shear—When a reinforced-concrete beam is subjected to flexural action, diagonal tensile stresses are set up. If, in a beam not having web reinforcement, these stresses exceed the tensile strength of the concrete, failure of the beam will ensue. When web reinforcement, made up of stirrups, or of diagonal bars secured to the longitudinal reinforcement, or of longitudinal reinforcing bars bent up at several points, is used, new conditions prevail; but even in this case, at the beginning of loading, the diagonal tension developed is taken principally by the concrete, the deformations which are developed in the concrete permitting but little stress to be taken by the web reinforcement. When the resistance of the concrete to the diagonal tension is overcome at any point in the depth of the beam, greater stress is at once set up in the web reinforcement.

For the composite structure of reinforced-concrete beams, an analysis of the web stresses, and particularly of the diagonal tensile stresses, is very complex; and when the variations due to a change from no horizontal tensile stress in the concrete at the remotest fiber to the presence of horizontal tensile stress at some point below the neutral axis are considered, the problem becomes more complex and indefinite. Under these circumstances, in designing, recourse is had to the use of the calculated vertical shearing stress as a means of comparing or measuring the diagonal tensile stresses developed, it being understood that the vertical shearing stress is not the numerical equivalent of the diagonal tensile stress, and even that there is not a constant ratio between them. It is here recommended that the maximum vertical shearing stress in a section be used as the means of comparison of the resistance to diagonal tensile stress developed in the concrete in beams not having web reinforcement.

Even after the concrete has reached its limit of resistance to diagonal tension, if the beam has web reinforcement, conditions of beam action will continue to prevail, at least through the compression area, and the web reinforcement will be called on to resist only a part of the web stresses. From experiments with beams, it is concluded that it is safe practice to use only two-thirds of the external vertical shear in making calculations of the stresses that come on stirrups, diagonal web pieces and bent-up bars, and it is here recommended for calculations in designing that two-thirds of the external vertical shear be taken as producing stresses in web reinforcement.

Experiments bearing on the design of details of web reinforcement are not yet complete enough to allow more than general and tentative recommendations to be made. It is well established, however, that vertical members attached to or looped about horizontal members, inclined members secured to horizontal members in such a way as to insure against slip, and the bending of a part of the longitudinal reinforcement at an angle, will increase the strength of a beam against failure by diagonal tension, and that a well-designed and well-distributed web reinforcement may, under the best conditions, increase the total vertical shear carried to a value as much as three times that obtained when the bars are all horizontal and no web reinforcement is used. Where vertical stirrups are used without being secured to the longitudinal reinforcement, the force transmitted between longitudinal bar and stirrup must not be greater than can be taken through the concrete, and care must be taken to provide for the larger bond stress developed in the longitudinal bars with this construction than exists in the absence of stirrups. Sufficient bond resistance between the concrete and the stirrups or diagonals must be provided. Where the longitudinal bars are bent up, the points of bending of the several bars should be distributed along a portion of the length of the beam in such a way as to give efficient web reinforcement over the portion of the length of the beam in which it is needed. The higher resistance to diagonal tension failures given by unit frames having the stirrups and bent-up bars securely connected together both longitudinally and laterally is worthy of recognition. It is necessary that a limit be placed on the amount of shear which may be allowed in a beam; for when web reinforcement sufficiently efficient to give very high web resistance is used, at the higher stresses the concrete in the beam becomes checked in such a way as to endanger its durability as well as its strength.

The section to be taken as the critical section in the calculation of shearing stresses will generally be the one having the maximum vertical shear, though experiments show that the section at which diagonal tension failures occur is not just at a support, even though the shear at the latter point be much greater.

The longitudinal spacing of stirrups or diagonal members, or the distribution of the points of bending of adjacent bent-up bars, should not exceed three-fourths the depth of the beam.

It is important that adequate bond strength or anchorage be provided to develop fully the assumed strength of all web reinforcement.

It should be noted that it is on the tension side of a beam that diagonal tension develops in a critical way, and that the proper connection must always be made between stirrups or other web reinforcement and the longitudinal tension reinforcement, whether the latter is on the lower side of the beam or on its upper side. Where negative moment exists, as is the case near the supports in a continuous beam, web reinforcement, to be effective, must be looped over, or wrapped around, or be connected with, the longitudinal tension reinforcing bars at the top of the beam, in the same way as is necessary at the bottom of the beam at sections where the bending moment is positive and the tension reinforcing bars are at the bottom of the beam.

Inasmuch as the smaller the longitudinal deformations in the horizontal reinforcement are, the less the tendency for the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by arranging and proportioning the horizontal reinforcement so that the unit-stresses at points of large shear shall be relatively low.

Where pure shearing stress occurs, or shearing stress combined with but a small amount of tensile stress in the concrete, as when a concentrated load rests on a slab, or other forms of punching shear are produced, or in the case of compression pieces, the element of tension will not need consideration, and the permissible limit of the shearing stress will be higher than the allowable limit when this stress is used as a means of comparing diagonal tensile stress.

Columns—By columns are meant compression members of which the ratio of unsupported length to least width exceeds about six, and which are provided with reinforcement of one of the forms hereafter described.

It is recommended that the ratio of unsupported length of column to its least width be limited to 15.

The effective area of the column shall be taken as the area within the protective covering; or, in the case of hooped columns or columns reinforced with structural shapes, it shall be taken as the area within the hooping or structural shapes.

Columns may be reinforced by longitudinal bars, by bands, hoops or spirals, together with longitudinal bars, or by structural forms which in themselves are sufficiently rigid to act as columns. The general effect of closely spaced hooping is greatly to increase the "toughness" of the column and its ultimate strength, but hooping has little effect on its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of "toughening" are adequately provided by a moderate amount of hooping, a larger amount serving

mainly to increase the ultimate strength and the possible deformation before ultimate failure.

Composite columns of structural steel and concrete in which the steel forms a column by itself, should be designed with caution. To classify this type as a concrete column reinforced with structural steel is hardly permissible, as the steel will generally take the greater part of the load. When this type of column is used, the concrete should not be relied on to tie the steel units together or to transmit stresses from one unit to another. The units should be adequately tied together by tie-plates or lattice bars, which, together with other details, such as splices, etc., should be designed in conformity with standard practice for structural steel. The concrete may exert a beneficial effect in restraining the steel from lateral deflection, and also in increasing the carrying capacity of the column. The proportion of load to be carried by the concrete will depend on the form of the column and the method of construction. Generally, for high percentages of steel, the concrete will develop relatively low unit-stresses, and caution should be used in placing dependence on the concrete.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

(a) Columns with longitudinal reinforcement only, to the extent of not less than 1% and not more than 4%; the unit-stress recommended for axial compression in "Working Stresses."
(b) Columns with reinforcement of bands, hoops or spirals, as hereinafter specified: stresses 20% higher than given for (a), provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.
(c) Columns reinforced with not less than 1% and not more than 4% of longitudinal bars, and with bands, hoops, or spirals, as hereinafter specified: stresses 45% higher than given for (a), provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.

The foregoing recommendations are based on the following conditions:

In all cases, longitudinal reinforcement is assumed to carry its proportion of stress. The hoops or bands are not to be counted on directly as adding to the strength of the column.

Bars composing longitudinal reinforcement shall be straight, and shall have sufficient lateral support to be securely held in place until the concrete has set.

Where hooping is used, the total amount of such reinforcement shall be not less than 1% of the volume of the column enclosed. The clear spacing of such hooping shall be not greater than one-sixth of the diameter of the enclosed column, and preferably not greater than one-tenth, and in no case more than 2½ in. Hooping is to be circular, and the ends of bands must be united in such a way as to develop their full strength. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered. The strength of hooped columns depends very much on the ratio of length to diameter of hooped core, and the strength due to hooping decreases rapidly as this ratio increases beyond five. The working stresses recommended are for hooped columns with a length of not more than eight diameters of the hooped core.

Bending stresses due to eccentric loads and lateral forces must be provided for by increasing the section until the maximum stress does not exceed the values above specified; and, where tension is possible in the longitudinal bars, adequate connection between the ends of the bars must be provided to take this tension.

Reinforcing for Shrinkage and Temperature Stresses—To be most effective, reinforcement (in amount generally not less than one-third of 1%) of a form which will develop a high bond resistance should be placed near the exposed surface and be well distributed.

WORKING STRESSES

General Assumptions—The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress to be allowed on concrete, we should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class but composed of different materials may have approximately the same degree of safety.

The following recommendations as to allowable stresses are given in the form of percentages of the ultimate strength of the particular concrete which is to be used; this ultimate strength is to be that developed in cylinders 8 in. in diameter and 16 in. long., made and stored under laboratory conditions, at an age of 28 days. In the absence of definite knowledge, in advance of construction, as to just what strength may be expected, the Committee submits the following values as those which should be obtained with materials and workmanship in accordance with the recommendations of this report.

Although occasional tests may show higher results than those here given, the Committee recommends that these values should be the maximum used in design.

TABLE OF STRENGTHS OF DIFFERENT MIXTURES OF CONCRETE

(In pounds per square inch.)

Aggregate	1:1:2	1:1½:3	1:2:4	1:2½:5	1:3:6
Granite, trap rock	3300	2800	2200	1800	1400
Gravel, hard limestone and hard sandstone	3000	2500	2000	1600	1300
Soft limestone and sandstone	2200	1800	1500	1200	1000
Cinders	800	700	600	500	400

Bearing—When compression is applied to a surface of concrete of at least twice the loaded area, a stress of 32.5% of the compressive strength may be allowed.

Axial Compression—For concentric compression on a plain concrete column or pier, the length of which does not exceed 12 diameters, 22.5% of the compressive strength may be allowed.

For other forms of columns the stresses obtained from the ratios already given under "Columns."

Compression in Extreme Fiber—The extreme-fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses, may be allowed to reach 32.5% of the compressive strength. Adjacent to the support of continuous beams stresses 15% higher may be used.

Shear and Diagonal Tension—In calculations on beams in which the maximum shearing stress in a section is used as the means of measuring the resistance to diagonal tension stress, the following allowable values for the maximum vertical shearing stress are recommended:

(a) For beams with horizontal bars only and without web reinforcement calculated by Formula (22): 2% of the compressive strength.

(b) For beams thoroughly reinforced with web reinforcement: the value of the shearing stress calculated as for a (that is, using the total external vertical shear in Formula (22) for shearing unit-stress), must not exceed 6% of the compressive strength. The web reinforcement, exclusive of bent-up bars, in this case, shall be proportioned to resist two-thirds of the external vertical shear in Formulas (24) or (25).

(c) For beams in which part of the longitudinal reinforcement is used in the form of bent-up bars distributed over a portion of the beam in a way covering the requirements for this type of web reinforcement: the limit of maximum vertical shearing stress (the stress calculated as for a), 3% of the compressive strength.

(d) Where punching shear occurs, that is, shearing stress uncombined with compression normal to the shearing surface, and with all tension normal to the shearing plane provided for by reinforcement: a shearing stress of 6% of the compressive strength may be allowed.

Bond—The bond stress between concrete and plain reinforcing bars may be assumed at 4% of the compressive strength, or 2% in the case of drawn wire.

Reinforcement—The tensile or compressive strength in steel should not exceed 16,000 lb. per sq.in.

In structural-steel members, the working stresses adopted by the American Railway Engineering Association are recommended.

Modulus of Elasticity—The value of the modulus of elasticity of concrete has a wide range, depending on the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that in computations for the position of the neutral axis and for the resisting moment of beams and for the compression of concrete in columns it be assumed as:

(a) One-fifteenth of that of steel, when the strength of the concrete is taken as 2200 lb. per sq.in. or less.
(b) One-twelfth of that of steel, when the strength of the concrete is taken as greater than 2200 lb. per sq.in., or less than 2900 lb. per sq.in., and
(c) One-tenth of that of steel, when the strength of the concrete is taken as greater than 2900 lb. per sq.in.

Although not rigorously accurate, these assumptions will give safe results. For the deflection of beams which are free to move longitudinally at the supports, in using formulas for deflection which do not take into account the tensile strength developed in the concrete, a modulus one-eighth of that of steel is recommended.

Suggested Formulas for Reinforced-Concrete Construction

(a) STANDARD NOTATION

1. Rectangular Beams.

The following notation is recommended:

f_s = Tensile unit-stress in steel,
f_c = Compressive unit-stress in concrete,
E_s = Modulus of elasticity of steel,
E_c = Modulus of elasticity of concrete,
$n = \frac{E_s}{E_c}$,
M = Moment of resistance, or bending moment in general,
A = Steel area,
b = Breadth of beam,
d = Depth of beam to center of steel,
k = Ratio of depth of neutral axis to effective depth d,
z = Depth of resultant compression below top,
j = Ratio of lever arm of resisting couple to depth d,
jd = d — z = Arm of resisting couple,
p = Steel ratio (not percentage).

2. T-Beams.

b = Width of flange,
b′ = Width of stem,
t = Thickness of flange.

3. Beams Reinforced for Compression.

A′ = Area of compressive steel,
p′ = Steel ratio for compressive steel,
f_s = Compressive unit-stress in steel,
C = Total compressive stress in concrete,
C′ = Total compressive stress in steel,
d′ = Depth to center of compressive steel,
z = Depth to resultant of C and C′.

4. Shear and Bond.

V = Total shear,
v = Shearing unit-stress,
u = Bond stress per unit area of bar,
o = Circumference or perimeter of bar,
Σ_o = Sum of the perimeters of all bars.

5. Columns.

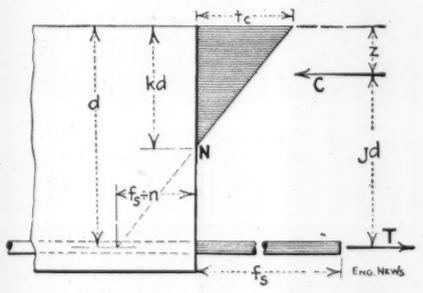
A = Total net area,
A_s = Area of longitudinal steel,
A_c = Area of concrete,
P = Total safe load.

(b) FORMULAS

1. RECTANGULAR BEAMS.

Position of neutral axis,

$$k = \sqrt{2pn + (pn)^2} - pn \qquad (1)$$



Arm of resisting couple,

$$j = 1 - \tfrac{1}{3} k \qquad (2)$$

(For f_s = 15,000 to 16,000, and f_c = 600 to 650, k may be taken at $\frac{7}{8}$.)

Fiber stresses,

$$f_s = \frac{M}{Ajd} = \frac{M}{pjbd^2} \qquad (3)$$

$$f_c = \frac{2M}{jkbd^2} = \frac{2pf_s}{jk} \qquad (4)$$

Steel ratio, for balanced reinforcement,

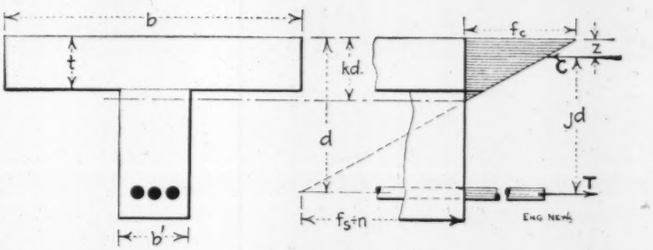
$$p = \frac{1}{2} \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)} \qquad (5)$$

2. T-BEAMS.

Case I. When the neutral axis lies in the flange: Use the formulas for rectangular beams.

Case II. When the neutral axis lies in the stem.

The following formulas neglect the compression in the stem:



Position of neutral axis,

$$kd = \frac{2ndA + bt^2}{2nA + 2bt} \qquad (6)$$

Position of resultant compression,

$$z = \frac{3kd - 2t}{2kd - t}\frac{t}{3} \qquad (7)$$

Arm of resisting couple,

$$jd = d - z \qquad (8)$$

Fiber stresses,

$$f_s = \frac{M}{Ajd} \qquad (9)$$

$$f_c = \frac{Mkd}{bt\left(kd - \frac{1}{2}t\right)jd} = \frac{f_s}{n}\frac{k}{1-k} \qquad (10)$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd = \sqrt{\frac{2ndA + (b - b')t^2}{b'} + \left(\frac{nA + (b - b')t}{b'}\right)^2} - \frac{nA + (b - b')t}{b'} \qquad (11)$$

Position of resultant compression,

$$z = \frac{\left(kdt^2 - \frac{2}{3}t^3\right)b + \left[(kd - t)^2\left(t + \frac{1}{3}(kd - t)\right)\right]b'}{t(2kd - t)b + (kd - t)^2 b'} \qquad (12)$$

Arm of resisting couple,

$$jd = d - z \qquad (13)$$

Fiber stresses,

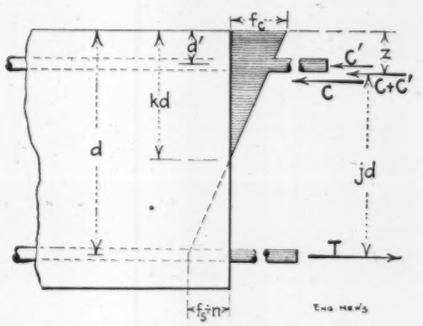
$$f_s = \frac{M}{Ajd} \qquad (14)$$

$$f_c = \frac{2Mkd}{[(2kd - t)bt + (kd - t)^2 b']jd} \qquad (15)$$

3. BEAMS REINFORCED FOR COMPRESSION.

Position of neutral axis,

$$k = \sqrt{2n\left(p + p'\frac{d'}{d}\right) + n^2(p + p')^2} - n(p + p') \qquad (16)$$



Position of resultant compression,

$$z = \frac{\frac{1}{3}k^3 d + 2p'nd'\left(k - \frac{d'}{d}\right)}{k^2 + 2p'n\left(k - \frac{d'}{d}\right)} \qquad (17)$$

Arm of resisting couple,

$$jd = d - z \qquad (18)$$

Fiber stresses,

$$f_c = \frac{6M}{bd^2\left[3k - k^2 + \frac{6p'n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]} \qquad (19)$$

$$f_s = \frac{M}{pjbd^2} = nf_c\frac{1-k}{k} \qquad (20)$$

$$f'_s = nf_c\frac{k - \frac{d'}{d}}{k} \qquad (21)$$

4. SHEAR, BOND, AND WEB REINFORCEMENT.

In the following formula, Σ_0 refers only to the bars constituting the tension reinforcement at the section in question, and jd is the lever arm of the resisting couple at the section.

For rectangular beams,

$$v = \frac{V}{bjd} \tag{22}$$

$$u = \frac{V}{jd\Sigma_0} \tag{23}$$

(For approximate results, j may be taken at $\frac{7}{8}$.)

The stress in web reinforcement may be estimated by the following formulas:

Vertical web reinforcement,

$$P = \frac{Vs}{jd} \tag{24}$$

Web reinforcement inclined at 45° (not bent-up bars),

$$P = 0.7\,\frac{Vs}{jd} \tag{25}$$

in which P = stress in single reinforcing member, V = amount of total shear assumed as carried by the reinforcement, and s = horizontal spacing of the reinforcing members.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For T-beams,

$$v = \frac{V}{b'jd} \tag{26}$$

$$u = \frac{V}{jd\Sigma_0} \tag{27}$$

(For approximate results, j may be taken at $\frac{7}{8}$.)

5. COLUMNS.

Total safe load,

$$P = f_c\,(A_c + nA_s) = f_cA\,(1 + (n - 1)\,p) \tag{28}$$

Unit stresses,

$$f_c = \frac{P}{A\,(1 + (n - 1)\,p)} \tag{29}$$

$$f_s = nf_c \tag{30}$$

The Emergency Use of Hypochlorite Water Disinfection at Philadelphia

By Francis D. West*

During the latter part of November, 1911, at a time when the Schuylkill River was very badly polluted, a break occurred in two of the filtered-water mains; as a result it was necessary for a time to pump water directly from the Schuylkill River to supply about 50,000 consumers. In spite of warnings to boil the water, an epidemic of typhoid fever broke out and over 300 cases resulted.

This epidemic, unfortunate as it was, caused the authorities to take such precautions that, following a second break, they succeeded in taking care of the situation in such a manner that not a single case of typhoid fever for the five weeks following has been directly attributable to the water.

The second break occurred about midnight of Oct. 25, about 100 yd. from the auxiliary pumping station of the Upper Roxborough Filter plant. The pipe cracked at a joint where new earth had been placed over and under it, following the bad break of 1911. The break was repaired in 20 hr. Meanwhile it was necessary to turn unfiltered water into the district. The precautions taken were so thorough and complete in every detail that they may well be copied by other cities finding themselves in a similar predicament. They were as follows:

(1) A emergency hypochlorite plant using three parts per million* of bleach (chloride of lime) corresponding to about 1.1 part of available chlorine @ 36%, was set up at the pumping station. The bleach was applied at the pump wells and consequently no water that had not been treated reached the consumer.

(2) Circulars printed for the purpose warning the people to boil the water were distributed by daybreak of Oct. 26.

(2) Posters were fastened to telegraph poles, displayed in store windows and similar places.

(4) The Bell and Keystone Telephone Companies notified all their subscribers—about 13,000.

(5) Announcement of the danger was made by all the newspapers.

(6) Special delivery letters were sent to all ministers in the district, requesting that they make announcement from the pulpit. Similar announcement was made at the schools.

(7) All fountains and public or semi-public water-supplies were shut off.

(8) Bacterial tests were made the day of the break and were followed by a thorough sampling of the district. This has continued since and a study of the results will be found further on.

(9) When the repairs had been completed and filtered water again turned into the mains, bleach was added at the filtered-water basin to thoroughly disinfect the pipes and pipe deposits.

(10) All fire plugs or hydrants were flushed out with the idea of stirring up and getting rid of all pathogenic bacteria as soon as possible.

The area receiving Schuylkill River water from the Upper Roxborough plant comprises a part of Wards 21, 22, 38 and 42, which comprise a part of the 5th and 6th districts, consisting of Germantown, Chestnut Hill, Roxborough and Manayunk, with a population of about 50,000, using in the neighborhood of 20,000,000 gal. of water a day.

Table I shows the cases of typhoid fever in the 21, 22, 28 and 42 Wards for a corresponding period of 1911 and 1912, as well as the cases in the entire city during the same time. The total number of cases for the four wards was practically the same in 1911 and 1912, and represents what might be called the normal for this time of the year. The water used in these wards comes from Lower as well as from Upper Roxborough filter plant.

TABLE I—TYPHOID FEVER CASES IN THE FOUR WARDS AND IN THE WHOLE CITY

	1911		1912	
Week ending.	4 Wards	City	4 Wards	City
Oct. 26	1	31	1	30
Nov. 2	5	20	1	18
Nov. 9	3	21	1	10
Nov. 16	1	20	3	19
Nov. 23	0	23	2	22
Nov. 30	0	16	3	23
Total	10	131	11	122

The bacteriological results are interesting and instructive and are given somewhat at length in Tables II, III and IV. They bring out two things: First, the effect of large doses of bleach on badly polluted water and second, the length of time it took to flush out the bacteria introduced with the raw water (see particularly, Table IV., bacterial counts from dead-end samples).

TABLE II—CONDITION OF SCHUYKILL RIVER AT TIME OF BREAK

Date.	Shawmont spigot Bacteria per c.c.	Turbidity	Belmont intake
Oct. 24	220,000	500	140,000
Oct. 25	130,000	180	120,000
Oct 26a	6,100	50	58,000
Oct. 27a	7,300	36	28,000

a Treated with bleach.

*Chemist in Charge of Torresdale Laboratory, Philadelphia, Penn.

*For the first two days. Subsequent changes in the rate are given elsewhere in this article.

The untreated water at Shawmont had a count in excess of that at the Belmont intake due to the dilution of Wissahickon Creek which enters the Schuykill below Shawmont.

The count was about 70,000 on the 26th and 40,000 on the 27th. At this the efficiency of the treatment at the pump wells was 91% for the 26th and 82% for the 27th.

TABLE III—CONDITION OF WATER ON REACHING VARIOUS TESTING POINTS.

Bacteria per c.c. on Gelatin at 20° C.

Date.	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Oct. 26.....	....	1400a	510	....	28*	....
Oct. 27.....	5	1600a	10	2200a	17*	....
Oct. 28.....	2	1800	4	190	100	....
Oct. 29.....	5	370*	100	120	....	....
Oct. 30.....	....	810	82	60	6A	....
Oct. 31.....	8	230A	140	100	6	180
Nov. 1.....	5	140	38A	92	4	89
Nov. 2.....	4	150*	35A	61	4	88
Nov. 3.....	2	....	....	49*	....	....
Nov. 4.....	5	....	....	26	3	....
Nov. 5.....	8	52	14A	....	....	49
Nov. 6.....	8	....	....	....	....	....
Nov. 7.....	6	19	4	....	16	14
Nov. 9.....	11	16	4	....	10	7
Nov. 12.....	2	12	14	....	8	13
Nov. 14.....	4	14	4	....	5	2
Nov. 16.....	4	6	5	....	2	5

No. 1 = Upper Roxborough Filtered Water Basin.
No. 2 = Tap at Mt. Airy Pumping Station.
No. 3 = Germantown above Chelton Ave.
No. 4 = 5th District Office.
No. 5 = 5214 Germantown Ave.
No. 6 = Mt. Airy Public School.

a = B.Coli present in 1 c.c.
* = B.Coli present in 10 c.c.
A = atypical Coli in 10 c.c.

ALL OTHER TESTS COLI ABSENT in 1 and 10 c.c.

At Mt. Airy Pumping Station (abandoned) the line is used but little and the water was turbid for four days and the count remained high until Nov. 7.

At Germantown and Chelton Aves. where the mains are used extensively the water was turbid but one day following which the count dropped to rise again due to stirring up of deposits in the pipes. The water at that point had a count of 14 per c.c. by Nov. 5.

The efficiency of the bleach at the Mt. Airy Pumping Station was 98% on the 26th and 96% on the 27th and at 5th District Office for the 27th it was 97%.

It will be noticed, then, that at a point where considerable water is used, although the count dropped by the third day it became higher and did not run consistently low until 10 days after.

The results from 5214 Germantown Ave., supplied by Lower Roxborough, but located near the border line between the two plants, shows the effect of the water backing up from Upper Roxborough during the first few days. By Oct. 30 the count was 6 per c.c. and after this it remained low.

TABLE IV—CONDITION OF WATER AT VARIOUS DEAD ENDS.

Bacteria per c.c.

	Source.	Oct. 30	Nov. 4	Nov. 15
	5th District.			
1	6808 Ridge Ave	4	6	4
2	6936 Ridge Ave	6	8	6
3	9156 Ridge Ave	81	85	contam.
4	302 Lamonte St	55	12	2
5	6247 Lawnton St	130	100	12
6	632 Leverington St	45	48	8
7	511 E. Connaroe St	22	28	6
8	260 Sumac St	250	54	19
9	279 E. Rochelle St	220	92	8
10	506 E. Roxborough St	58	9	4
11	Vicarous and Osborne St	42	13	7
12	5200 Ridge Ave	32	17	4
13	5501 Ridge Ave	32	16	4
	6th District.			
14	Carpenter N. E. Emlen St	6	2	4
15	McCallum N.W. of Carpenter St	7	10	4
16	Germantown N.W. Johnson St	19	12	4
17	Morton N.W. of Johnson St	320	24	6
18	Germantown N.W. Allen's Lane	85	20	8
19	Gowen S.W. Stenton St	230	94	15
20	Mermaid S.W. of Stenton St	280	81	4
21	Mt. Pleasant S.W. of Emlen St	180	45	11
22	Willow Grove & Township Line	250	57	3
23	Upsal and Jefferson St	180	38	5
24	Stenton S.W. of New St	41	21	5
25	Highland S.W. of 30th St	160	56	4
26	3719 Emlen St	64	20	5
27	Greene N.W. of Carpenter St	33	9	28*

Tests for B. Coli communis Oct. 30 all negative 1 and 10 c.c., Nov. 4 all negative 1 c.c. Upsal and Jefferson positive on 10 c.c. and 506 E. Roxborough and 5200 Ridge Ave. atypical on 10 c.c. Nov. 15 all negative on 1 c.c. and 10 c.c.

The filtered water basin on each of these dates had a bacteria count of 5 per c.c.

Samples 1, 2, 14 and 15 were low on each date, it is thought that no turbid water reached these points. Nos. 8, 9, 17, 19, 20 and 22 are the most interesting. These show a drop from an average of 260 on Oct. 30 to 67 on Nov. 4 and 7 on Nov. 15.

Omitting Nos. 1, 2, 14 and 15 the other 23 have an average of 122 on Oct. 30, of 36 on Nov. 4 and 7 on Nov. 15

B. Coli Communis.

Omitting the tests of the first two days when the water was turbid and not filtered, we have the following results:

Total test of 1 and 10 c.c. = 133
Total positive tests of 1 c.c. = 0
Total tests of 10 c.c. positive = 3 = 2.3%
Total tests 10 c.c. atypical = 7 = 5.3%

It is very evident from these results that the bleach thoroughly disinfected the water.

By Nov. 17 is was stated by the writer that he believed the water to be entirely safe to drink. Samples tested since that date have had counts as low as 0 per c.c. and B. coli has been absent from all tests.

On Nov. 26, one month from the time of the break, the Bureau of Health notified the consumers that the water could be used with out danger.

The hypochlorite treatment has been continued.

The strength of the bleach used at Upper Roxborough was as follows:

Oct. 26 and 27, 3 parts per million (powder); Oct. 28, 29 and 30, 2½ parts. Oct. 31 to Nov. 11, 2 parts; Nov. 11 to date (Dec. 2) ½ part.

All the filter plants in Philadelphia have been adding bleach to water in the filtered water basins as an added precaution for over a year.

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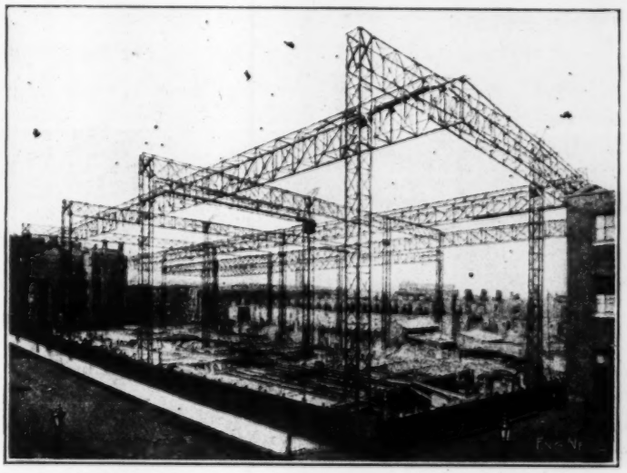
Steel Frame and Traveling Crane for Building Erection

The novel framework shown in the accompanying halftone is now being used in the erection of H. M. Stationery office, in London, Eng., a new reinforced-concrete structure. It is an adaptation of the common shipyard crane for the first time applied to building work on a large scale. The new building occupies the major portion of an entire city block, with outside dimensions of 106, 189, 377 and



STEEL FRAME CARRYING TRAVELING CRANES FOR ERECTING AN ENGLISH BUILDING

323 ft. on its four sides, and will be, when completed, eight stories high. The frame shown covers this entire area and allows a clearance for the complete height of the building.

In design the frame comprises four transverse rows of columns across the narrow length of the lot and five rows along the wide length, the columns being connected longitudinally by box frames upon which electric cranes are run, and transversely by bracing frames which are raised above the elevation of the longitudinal frames to allow clearance for the traveling cranes. The whole structure is of riveted steel work. The positive control of the whole area and the freedom from the necessity of constantly changing the erection cranes to avoid the rising structure are considered sufficient reasons for the use of this erection frame, which involves a rather high first cost.

The view shown herewith is taken from *Concrete and Constructional Engineering*, of London, Eng.

Lessons from the Failure of a Weir and Sluices on Porous Foundations

By W. G. Bligh*

The principles which underlie the design of any hydraulic work founded on a porous sub-stratum have been explained *in extenso* in previous articles by the author in Engineering News†, so that further detailed explanation will be unnecessary and it will suffice to state that the stability of structures built on a porous foundation of

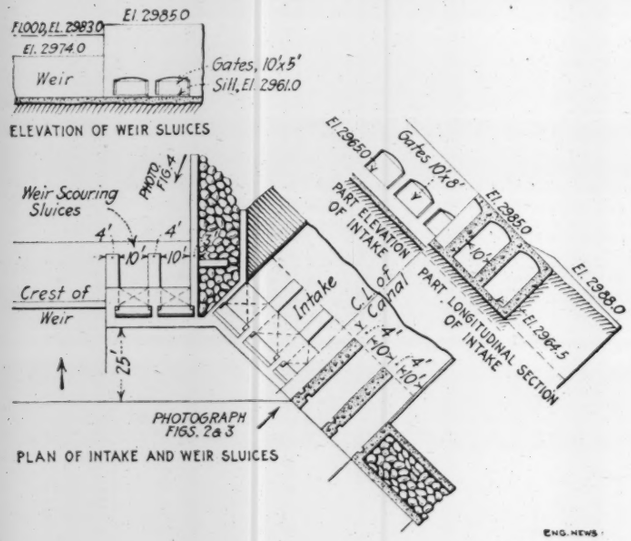


Fig. 1. Plan and Elevation of Weir and Intake, Which Failed

sand or boulders and subjected to the pressure of a head of water is dependent entirely on the ratio the length of the enforced percolation underneath the impervious superstructure bears to the head of water acting on the work. Symbolizing the former by the latter l and the latter by H the ratio $\frac{l}{H}$ or c is termed the "percolation factor."

The proper safe value of this factor is found, not by artificial experiments on a small scale, but by indisputable statistics which bear on the capacity of the various pervious materials which may compose a river-bed to resist the undermining influence of the pressure of the water upheld. The ratio $\frac{1}{c}$ is termed either the "hydraulic gradient" or the "virtual slope." The values of percolation factors for the various materials adopted by the writer and given in the articles above alluded to are:

Very Fine Sand and Silt	18
Fine Sand	15
Coarse Sand	12
Sand and Gravel	9
Boulders and Gravel	4 to 6

The statistics referred to are based on the failures of works which originally, from ignorance of the principle involved, were designed with too short an effective base, or length of percolation, for safety. Thus, suppose a case where a work fails by undermining, or what is technically termed 'piping," in which the ascertained value of c is say 10. When repaired this structure is provided with an increased travel of percolation, raising the ratio $\frac{l}{H}$ or c to, say, 18, we are now enabled to deduce the fact that 10 is too small a value for the "percolation factor," while on the other hand 18 is certainly a safe value, though it may be wastefully large. Examination of other works similarly situated, as regards quality of the pervious sub-stratum, enable a fairly correct value to be deduced and the real safe value for c may be found to be close to, say, 15. Based on this deduction from ascertained facts, any work can be designed with confidence as to its stability against undermining if its horizontal base width with its vertical depressions and sinuosities added in, i.e., l, equals $c \times H$. Under these circumstances any failure of a work on a porous foundation is often extremely valuable as furnishing an additional fact on which a reliable value for c can be deduced suitable for the particular description of sub-stratum in question.

Such a failure, from which some useful deductions may be made, recently occurred in an irrigation project which, for the present, shall be nameless. The details of the design, the collapse of a portion of the work and the lessons which may be learned from the collapse are given below.

The diversion works in question are located on a river which is split in two channels by an island, and both

Figs. 2 and 3. Views of Intake before and after Failure

channels are crossed by diversion weirs, connected by a riprapped embankment across the island. Each of the weirs is a little over 500 ft. in length and the embankment is 2000 ft. long. The canal takes off at one end of one of the weirs, Fig. 1, where are situated the weir scouring sluices, two openings 5x10 ft. each, and closely adjoining is the canal intake, consisting of five openings

*44 Dupont St., Toronto, Can.

†Engineering News, Dec. 29, 1910; p. 708 (erratum, Jan. 12, 1911, p. 52); Apr. 13, 1911, p. 444.

each 8x10 ft. The full supply of the canal is 8 ft. deep, with a discharge of 1500 sec. ft.

The river is supposed to have a maximum discharge of 118,000 sec. ft., which will cause an afflux of about 9 ft. above the crest of the south weir. The principal levels are as follows:

Crest of south weir	El. 2974
Crest of north weir	2975
Low water level	2960
Flood afflux	2983
Sill of weir sluice	2961
Sill of intake sluice	2965

The maximum statical head acting on the weir occurs when the water upstream is at crest level and the channel below at low-water level, viz. 14 ft. (Fig. 6.) The maximum head acting on the intake occurs when the river is in full flood and the canal below empty and is then 18.5 ft. (Fig. 9.)

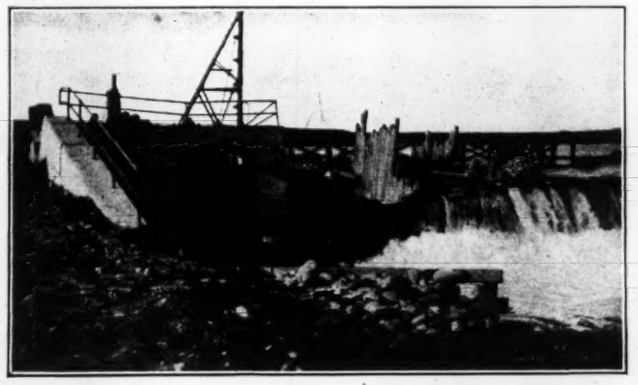


FIG. 4. VIEW LOOKING UPSTREAM AT WEIR SLUICES AFTER FAILURE
(Position of view shown on Fig. 1)

The materials of which the bed of the river is composed consist of boulders, gravel and sand. A safe value for the percolation factor c should, it is deemed, be not less than 6, consequently the requisite length of travel including any vertical depressions beneath the weir or the weir sluices should be $14 \times 6 = 84$ ft., while for the intake the value of l should be $18.5 \times 6 = 111$ ft.

On analysis it will be found that the weir proper falls short of this, but the other works were provided with a sufficient length of travel, provided the fore aprons had been made thick enough to withstand the uplift, which is very considerable. This they were by no means capable of doing, consequently with a comparatively low head they blew up and the fore aprons once destroyed, the length of travel of the percolation was reduced to a figure much below 84 ft., so that the works were immediately undermined and collapsed bodily.

Fig. 2 shows the intake before destruction and Fig. 3, taken from about the same location, shows its condition after the settlement of the foundation. Fig. 4 is a view of the weir sluiceways after the collapse.

ALIGNMENT OF WEIR SLUICES

With regard to the alignment of the work, the function of weir scouring sluices is to maintain a clear channel in front of the canal intake. To effect this it is requisite that the latter be situated close adjoining to the sluice ways and be aligned at right angles to the axis of the weir. A further necessary adjunct is the provision of a "divide" wall with deep foundations between the weir and the approach and escape channels of the sluices.

This wall protects the weir from the consequence of any disaster occurring to the sluices, which are always a vulnerable point, and further promotes scour by confining the channel. In these works no divide wall whatever had been provided, with the natural consequence that the failure of the weir sluices carried with it a considerable length, viz., 40 ft., of the weir itself.

The alignment of the canal head being at an angle of 135°, not 90° with the axis of the weir, neutralizes the useful effect of the scouring sluices. Reference to Fig. 1 will clearly show that a triangular spit of gravel or sand is bound to be deposited in front of the gates of the intake, the prevention of which is the essential function of the weir sluices; their value therefore, owing to this wrong alignment, is largely discounted.

WEIR PROPER

Figs. 5-7 are profiles of the weir at different locations. From these figures it is apparent that the weir body consists of a solid triangular mass of concrete which is provided with rear and fore aprons of like material. The levels of these aprons and that of the base of the structure itself were made to coincide with the bed of the river channel as it existed at the time of construction. Wherever a dam is founded on solid bedrock, the carrying down of the profile in this way is to be commended as being the most economical method. When, however, as in this case the bed of the river is composed of loose material which can easily be shifted either forward or sideways by the action of water in a high state of commotion, as would naturally occur below an overfall weir, such a procedure is open to strong objection. In every river bed, some part or parts of the water section are deeper than the rest, forming a depressed channel which carries the greater part of the discharge. When, however, as is the case with an overfall weir, a level wall is built across the river, the volume passing over is practically the same at the ends of the crest as in the center, or wherever the deep channels exist.

These depressions therefore tend to become obliterated

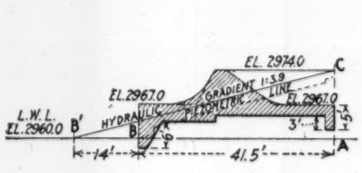


FIG. 5. MINIMUM SECTION

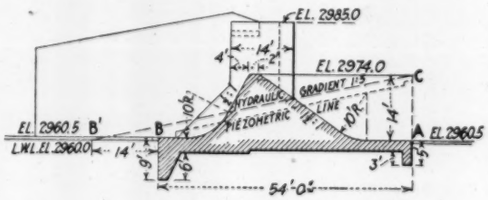


FIG. 6. SECTION ADJOINING SLUICES

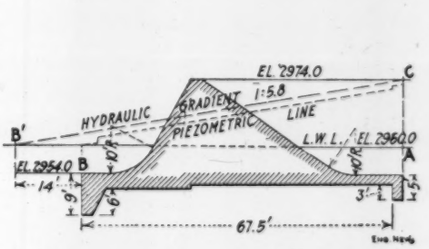


FIG. 7. MAXIMUM SECTION

FIGS. 5 TO 7. SECTIONS OF MAIN WEIR, SHOWING HYDRAULIC GRADIENT AND PIEZOMETRIC LINE

below the weir as they undoubtedly are in the upper reach and this process of leveling up takes place at the expense of the higher parts which become gradually denuded. This being the case it is distinctly erroneous to design the foundations on the assumption that the river bed will remain unchanged in profile as if it were rock, after the weir is built and the conditions of flow radically altered. The proper procedure in all such cases is undoubtedly to level up the bed artificially by excavating the higher parts and with this material filling up the hollows, the latter being protected by deep curtain walls to retain the fill. On the platform thus formed the concrete weir wall with its complements, the rear and fore aprons, which are placed generally at low-water level, is built.

Thus inexpensive fill takes the place with perfect safety of a solid concrete mass. Many examples of this construction exist in river weirs in the United States while in Indian works light friable sand is made to do duty in the same manner, all that is essential is the provision of sufficiently deep protective curtain walls, in order to prevent any possible movement of the inclosed fill.

The hydrostatic conditions being the same right through, i.e., a maximum head of 14 ft., the length of travel of the enforced percolation should also be the same throughout. Examination of the profiles in Figs. 5-7 shows a considerable variation in base width and consequently also in the values of c.

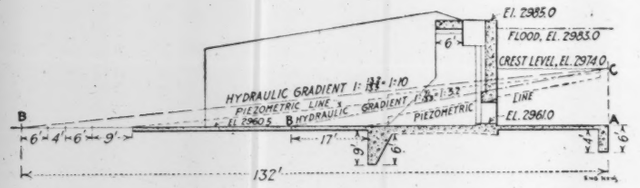


FIG. 8. CROSS-SECTION OF WEIR SLUICES, SHOWING HYDRAULIC GRADIENT AND PIEZOMETRIC LINE

Thus in Fig. 5 the hydraulic gradient works out to 1 in 3.9, in Fig. 6 to 1 in 5, and in Fig. 7 to 1 in 5.8. These grade lines are marked on each of the profiles, together with the parallel piezometric lines which demonstrate the height to which water would rise if piezometer pipes were inserted through the impervious base, and consequently the area contained between the piezometric line and the base represents the uplift in terms of the depth of water. If these graphical lines were drawn in on every design of a work subjected to uplift, mistakes such as are being chronicled would be impossible, for the reason that the extent of uplift and that of the virtual slope at once appeal to the eye. The piezometric line, which the writer was the first to introduce, when the base is only horizontal, coincides with the hydraulic gradient, but when vertical depressions occur in the base, it is stepped down at these points, and then the outline is lowered below the hydraulic grade line. Its application is most simple, and only requires a pencil, a parallel rule and a few grains of common sense to effect. Take any of the profiles, it will be seen that there is a point of ingress for the percolating water A and a point of egress B, the distance between these is the horizontal component of the length of travel of the percolating fluid.

To obtain the hydraulic grade line graphically, the line AB has to be lengthened to B^1 to the extent of the sum of the vertical depressions below the base line. The percolating current has been proved by experiment to cling to the sides of the vertical obstructions, the length of travel being measured down one side and up the other, except in such cases where the vertical obstructions are closely adjacent. In Figs. 5-7 the point of ingress is at A, that of egress is at B, the end of the fore apron, at low-water level. The line AB is prolonged to B^1, BB^1 being made equal to 14 ft., which is the sum of the vertical components of the travel, viz.: $5 + 3 + 6$ ft., the final one at the outside of the cut-off wall being omitted from consideration, as it varies in value in the different sections and if scour took place would be nonexistent. The effect of sheet piling below the fore curtain is also neglected. The line AC made equal to H, the head of 14 ft., is drawn vertically and CB^1 the hypothenuse of the triangle CB^1A is then the hydraulic gradient, which is $\frac{1}{H}$, or $\frac{55.5}{14} = 1$ in 3.9.

The piezometric line is formed as follows. In Fig. 5 the vertical depression 6, 3 and 5 ft., respectively, are marked out from B to B^1, in reverse order; from the points thus obtained lines parallel to CB^1 are drawn intersecting verticals drawn up from the reciprocal vertical depressions in the base line. The piezometric line is thus stepped down wherever a vertical step down or up occurs in the base, the depth of each step being obviously equal to the actual vertical travel divided by c. Thus the depth of the initial step at C will be $\frac{5}{3.9} = 1.2$ nearly, the next will be $\frac{3}{3.9} = 0.75$, and so on. The graphical procedure, however, avoids any such calculation, the same result being produced automatically by the method described.

In the maximum profile, Fig. 7, there is clearly great waste of expensive concrete whose place could well have been taken by protected boulder and gravel fill, whereas in Fig. 5 the whole profile lies above low-water level exposed to having its foundations eroded and deposited in the deep central channel, while its hydraulic grade is only 1 in 3.9 instead of 1 in 6. Its condition is therefore considered decidedly unsafe and it only requires a very low river or a high flood to bring about a catastrophe. The only hope lies in a thick deposit of silt taking place upstream, forming a natural impervious rear apron which will throw the ingress point of the line of percolating travel further back and so flatten out the hydraulic gradient to a safe ratio. This natural action of silt-laden water has saved many an ill designed work from inevitable destruction by undermining. It cannot, however, be depended upon and in this case unfortunately the river is not much of a silt bearer.

The outline of the profile of the weir is distinctly crude. It is that of a trapezium with a crest width of 2 ft., increasing in thickness in the ratio of $\frac{2}{1}$, whereas this slope should be more in the relation of $\frac{2}{3}$. There is, therefore, a surplusage of material which is wholly useless. Such a profile is only suitable for a hollow dam, similar to the Ambursen type, the weight of the material in which is insufficient to withstand the horizontal water moment without assistance. The additional weight required is obtained by sloping the back so as to cause it

to support a mass of water as well, and further insures a wide base. What is then suitable under totally different conditions becomes wasteful when applied to a solid dam. Where material would be of value is in the horizontal aprons which are alarmingly deficient in length. The rear apron requires widening to bring the hydraulic grade down to 1 in 6, while the fore apron has a width of only 12 ft. which undoubtedly requires supplementing, not however by an impervious floor, but large blocks of concrete laid with open joints, as was done in the case of the Granite Reef dam on the Salt River, Ariz.*

A properly designed solid weir under prevailing conditions would have a crest width of 6 ft. and a base width of about 13 ft., which will be just half the superficial area of this profile at a depth of 14 ft. The most suitable design, however, would probably be of the buttressed-slab or buttressed-arch type.

Weir Sluice

A section of the weir sluice is given in Fig. 8. The rear apron of the weir, the only commendable feature in that crude design, has been carried at a reduced thickness right across the rear of the weir sluice till it runs into that of the intake (Fig. 1). The latter is therefore practically devoid of a rear apron, two bays being without any such protection. Under these circumstances, inspection of Figs. 7, 8 and 9 will show, that under a head of 11 or of 9 ft., the uplift is greater beneath the canal intake than that acting on the weir sluices. The great advantage of a rear apron is apparent in these figures. It throws back the incidence of the point of ingress and the perpendicular of the triangle CAB^1, thus flattening the hydraulic grade and reducing the uplift. A line of deep sheet piling would have a similar effect, but in a boulder formation an impervious diaphragm is impossible of realization since even with the introduction of sheet-steel piling the presence below the surface of large boulders will cause the separation and bending of the piles, leaving gaps. For this reason the sheet-piling which exists at the end of the fore apron of the weir has been omitted from consideration, as having no statical value. A fore apron or fore sheet piling has the opposite effect to the rear apron, as it tends to throw the point of egress farther outward, thus increasing the uplift.

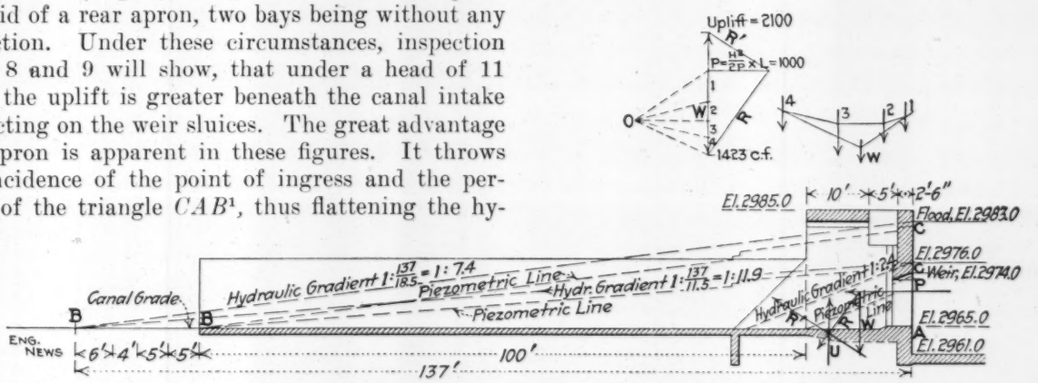


Fig. 9. Cross-section through Intake, Showing Hydraulic Gradient and Piezometric Line

Intake

With remarkable fatuity a fore apron 1 ft. thick no less than 100 ft. long had been added on to the intake. The fatal error was the immediate cause of the catastrophe, the uplift, even with a low head of 11 ft. or 9 ft., with water only at crest level, was far too great for a thin floor to stand and naturally it blew up with great force not only beyond the intake head but as far as could be seen the floor situated between the piers themselves was also upset. When this occurred the hydraulic gradient suddenly fell back from 1 in 12 to 1 in 2.4, a value of c far too low and in addition a vacuum was formed which accelerated the complete undermining of the structure. The connection between the intake and the weir is extremely weak (Fig. 1), consisting of only two shallow walls, probably at different base levels without any floor between, and toward the upstream the only protection is a wall 2½ ft. thick with a portion of rear apron and nothing behind it except a backing of boulders. This was speedily undermined and the collapse of the weir sluices and adjoining weir was brought about, by this means, if the weir sluice floor did not simultaneously blow up, as is the most likely case.

If the intake was wrecked under a head of 11 ft. or less it clearly could not have stood a moment under the enormous uplift which would occur at full-flood level. Three sets of grade and piezometric lines are shown in Fig. 9. The lowest represents the state of affairs after the blowing up of the fore apron, the middle one that occurring just before that event and the upper one the impossible condition, supposing the floor had resisted when the maximum head was applied. Both works tilted backward at an angle of 10° or 12°. This is due to the couple caused by the uplift and the weight of the structure, which for the full flood levels is graphically delineated. If the fore apron, which was directly responsible for the disaster, had not been built the intake would still have failed by undermining, the percolation factor being in that case no higher a value than 3, while that of the weir sluice is 5.2. At full flood levels the hydraulic gradient would be as steep as 1 to 1, in the case of the intake.

Had a sufficiently wide rear apron been provided, which could be composed of a thick layer of clay, covered with riprap and the fore apron or floor rendered pervious, the work would have been perfectly safe. It is an extremely massive construction, the piers being 4 ft. in thickness of solid concrete. In cases where the foundation is porous, weight in a structure except as regards overturning moment, is of little value. It certainly has the property of retarding disintegration of the loose substratum, but failure must occur sooner or later by undermining, if the hydraulic gradient is too steep. The work is now a complete ruin and has been blown up to remove the gates.

Repairs

A new intake will be built at another site, as the river is wide enough at this part to allow plenty of space for the purpose. Weir sluices will be of no use and the

*Engineering News, Oct. 1, 1908, p. 366; Jan. 7, 1909, p. 1.

scouring will have to be accomplished through the intake itself, utilizing an escape head which is built lower down as outlet. The weir will be shortened by 40 ft., as it is proposed to build a south abutment at the point where the fracture took place.

Port Angeles Dam

The accounts of the accident to the dam at Port Angeles, Wash., contained in Engineering News, Dec. 5, and 26, 1912, are of interest as a demonstration of how little the principles governing the design of dams and weirs built on a porous foundation are understood.

The head of water upheld in this case is 100 ft. The length of travel is as follows: horizontal, 197 ft.; vertical, $2 \times 10 = 20$ ft. (around the sheet piling), or 217 ft. total. The head being 100 ft., the hydraulic gradient will be 1 in 2.17.

A percolation factor of a little over 2 is absurdly inadequate for gravel; it is considered that 6 is the very minimum allowable value. This increased length of travel can be easily provided by simply lengthening the rear apron of puddle or clay 323 ft. and also providing sheet piling 40 ft. deep underneath the dam. That in front of the dam is clearly of no use whatever, in fact, if effective it would be positively detrimental as increasing the uplift. The new length of travel would then be 600 ft. made up of 520 ft. horizontal and 80 ft. vertical travel. The profile of the wall could then be largely reduced in sectional area. The profile as it stands is wasteful and primitive. If a core wall reaching down to rock and forming a watertight connection with the same on all sides were built it would obviate any failure by undermining. The uplift, however, would be very great.

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The Semi-centenary of Underground Rapid Transit railways was celebrated in London, on Jan. 10, of this year. Prior to 1863 and as far back as 1844, a number of the various railways entering London had stations within the city limits between which they operated a passenger service, but in every case the lines were merely the termini of the main railroads running out of the city. As early as 1846, however, the overcrowding of the London streets and the consequent difficulty and danger of locomotion began to be a matter for public discussion. New lines of streets were formed and fresh channels of communication opened, but the traffic continued to be greater than the service.

At length, in 1846, a parliamentary committee examined and reported upon a number of different projects to remedy the difficulty. About 20 schemes were submitted to this committee, most of which were merely intended to extend the existing roads. A few, however, proposed to lay new local lines either in what they termed railway streets, that is, new thoroughfares to be formed by wholesale demolition of buildings, or in open cuts which meant the conversion of streets into trenches, or on viaducts. One proposal, that by Charles Pearson, then the city solicitor of London, suggested that a new line be built into the crowded district of London as a shallow subway. This project was intended to have six tracks in the subway with an 80-ft. road above. Mr. Pearson's suggestion was not carried out, but he was engaged to work out a system of metropolitan subways somewhat on the same plan as the one which he suggested. As a result of his study, in 1859 parliamentary permission was obtained to build what afterward became the first underground rapid transit subway under the direction of the Metropolitan Ry. The first section of this road was opened on Jan. 10, 1863.

This subway was built as a single arch tunnel carrying four broad-gage tracks with multiple-car trains drawn by a steam locomotive. So great was the success of the short line first built that immediately thereafter plans were formulated for the construction of a much more extensive line, which eventually developed into the present Metropolitan District Ry. A view of one of the first trains and a short history of the railway appears in the "Engineer," of London, for Jan. 10, 1913.

Corrosion of Spikes in Ties Treated with Zinc-Chloride*

In the past 12 or 15 years the use of the zinc-chloride process for treating ties has extended to numerous railways covering the territory west of Chicago, and with this largely extended use of the process there have been instances where serious corrosion of spikes (and tie-plates also) has occurred. Such damage, however, is by no means general. On some roads, where such ties have been in use in large numbers for many years, there is no evidence of corrosion of track fastenings (either in hard-wood or soft-wood ties). On other roads, the corrosion appears in some localities, while in others it is absent.

The cause of the corrosion is not well understood. In the earlier days it was laid to the presence of free acid in the treating solution, and this may be responsible for cases of recent occurrence. The treatment of ties is now done more largely by contracts and, there may be more hasty methods of handling the ties, with possibly less attention to the purity of the solution. The remedy is adequate inspection at the treating plants by representatives of the railways.

In the Wellhouse process, a weak solution of glue and tannin or tannic acid is either mixed with the chloride or is applied later as a separate solution, in order to plug up the cells and prevent the entrance of moisture to leach out the zinc-chloride salt, which is easily soluble. It has been claimed that no corrosion of spikes has been noticeable where glue and tannin have been used. On some roads it has been noticed that while steel spikes have been appreciably corroded in zinc-treated ties, iron spikes have shown no evidence of corrosion.

The most widely prevailing opinion as to the cause of corrosion has reference to the seasoning of the ties after treatment. It is thought that if zinc-treated ties are immediately put into the track the spikes will corrode, but that if the ties are permitted to dry out, so that all the salt has solidified, there will be no appreciable corrosion. It is an interesting fact that in all cases of corrosion the action on the spike is most intense at the face of the tie. Others claim that the supposed greater activity of the corrosion in freshly treated ties is due to galvanic action, and that when once the solution in the tie becomes evaporated, the dry salt will not act upon the spike or other fastening. Contrary to this view is the fact that spikes in tracks at the doors of tie-treating cylinders, constantly exposed to the dripping of zinc-chloride for periods, as long as ten years, show no evidence of corrosion, while spikes in untreated ties in main track (in the same neighborhood) in service the same length of time, have become badly corroded. However this may be, it is in line with good practice to give treated ties time to season before putting them into the track, as the zinc-chloride, once solidified, is less liable to be leached out of the tie by the dampness of the ballast.

On electric railways the corrosion of spikes is more frequently met with than on steam railways, and it is thought, that it is more rapid in zinc-treated ties. It seems to be agreed that the corrosive action is caused by electrolysis from the currents which escape to the ground from the return circuit of the track. In the case of ties treated with a metallic salt the path to the ground would naturally be easier than through untreated ties. There is the question of a similar action on the spikes from the track circuits of electric block signals. Inquiry, however, fails to show that any case of spike corrosion can be traced to the signal circuits. It might be worth while to investigate whether the corrosion of spikes in steam railway tracks might not in some cases be caused by stray ground currents originating in an intersecting or parallel track of an electric railway.

On a railway system where corrosion of steel spikes in zinc-treated ties had occurred, experiments were undertaken to determine the relative corrosive effect on spikes driven in untreated ties, in ties treated with zinc-chloride and in ties treated with water. After seven years of service the spikes were pulled out and it was found that corrosion had taken place with all the spikes under the three test conditions, and that the differences were so slight that conclusions in favor of any one of the three seemed unwarrantable. On some roads where corrosion has occurred it has been found difficult to determine whether or not the corrosion in treated ties was more than in the untreated ones. This makes it certain that some other agency than zinc-chloride was responsible for at least some of the corrosion. On other roads, inspection has shown that corrosion of spikes on zinc-treated ties was caused by the brine drippings from refrigerator cars.

*From a report by W. M. Camp, published in the "Bulletin" of the Roadmasters and Maintenance-of-Way Association.

EDITORIALS

The semiannual INDEX to ENGINEERING NEWS, that for Vol. 68, July-December, 1912, is mailed with this issue. All subscribers who fail to receive a copy should notify this office as soon as possible.

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The poor old horsepower, like the hound dog in the campaign song of last fall, is being kicked around in a most unfeeling way of late. As noted in these columns a week ago, the myriawatt (10,000 watts) is being blandly offered to the engineering public as altogether better than the horsepower. It's almost the same as a "boiler horsepower" anyway, say its sponsors, so why not shove the horsepower back out of sight?

And now comes the Bureau of Standards with a proposal to make the horsepower an even 746 watts—a proposal which is about equivalent to trimming the ears of the old hound to make him look like an uptodate bull dog. These would-be surgeons do not realize that the hound is too old to stand such an operation. The chances are after he had been trimmed he would not know where he was at and his old friends would have no further use for him.

Fortunately for the horsepower, like the hound dog, he has some staunch friends left, who will protest against his being kicked around. It will be a long time, we believe, before the term horsepower disappears from commercial usage. The electrical engineers can think in watts, and so probably can numbers of other engineers trained in the technical schools; but to the average man the horsepower has a concrete meaning which the watt cannot possibly have. There is no objection, of course, to the kilowatt or to the myriawatt, for that matter, if the natural process of evolution shall establish that new unit as a definite breed; but let us not imagine that by simply kicking the horsepower out of the back door he is going to disappear. Old hounds have a surprising vitality, and our guess is that he will be able to follow the trail for many a long day yet.

❖

Engineer Members for the New York Public Service Commissions

The American Institute of Consulting Engineers deserves the approbation of the profession for the aggressive attitude it has taken in urging upon Governor Sulzer of New York that engineers should be selected to fill the existing vacancies in the New York Public Service Commission. A delegation of Institute members went to Albany, on Jan. 22, and interviewed Governor Sulzer and also left with him a letter setting forth the reasons why engineers should be selected for these positions.

Of even greater importance perhaps, because of its influence upon public opinion, is the fact that the Institute took measures to give proper publicity to its action. The New York daily papers of Jan. 30 and 31 gave prominent space to this appeal from the Institute representatives. The following quotation from the argument by the Institute's representatives before Governor Sulzer is well worth putting on record in these columns:

> At a recent meeting of the American Institute of Consulting Engineers the undersigned Committee was appointed to present the views of that Institute, and what are probably the views also of a majority of the engineering profession on the subject of appointments to the Public Service Commissions of this State.
>
> We have no personal or individual ambitions to serve and no candidates to present, but as citizens, however, and expressing the sentiment of a body of professional men who, more than any other understand the problems involved in the execution and management of great public works, we may be permitted an altruistic viewpoint, and speak for what we believe to be the best interests of the city and the State.
>
> * * * * *
>
> It is unnecessary to here go into the history of the changing activities of the New York Commission, but had it been constituted as above indicated [with able engineers among its members] the preparation of unworkable and costly plans for the so called Triborough System, now largely abandoned, for which not a single bid for operation was received, would not have occurred. The plans for so much of the original routing as constitute a part of the dual system under consideration have, because of the activities of engineers, already been changed so as to effect a saving of many millions of dollars. With a commission, thoroughly informed on questions of construction and transportation, rapid transit would already have been an accomplished fact on terms creditable and fair to both city and the transportation companies.
>
> The new commissioners, whether chosen because of experience in executive and business affairs, or legal attainments and knowledge of corporation law, or special railway experience, training and knowledge, or wide engineering experience should be men of such character and standing as to command not only your own, but the public's confidence.
>
> This committee believes that the best interests of the community will be conserved, and the maximum of credit accrue to your administration, if you establish the practice obtaining elsewhere of selecting for the Public Service Commissions, men of broad engineering experience, not because of their profession as engineers, but because the problems continually confronted are best solved by men of character and executive capacity who have had engineering training.
>
> There are ample precedents for your guidance. The National Government selected engineers for its Canal Commissions and an army engineer officer to supervise the Panama project in its entirety; Chicago employed an engineering board to settle its financial, constructive and operating transportation problems, and another board of three engineers to determine its rapid transit system; the Pennsylvania Terminals, costing $125,000,000, was supervised by an engineering commission, and the road's president is an engineer; the New York Central's Terminal improvements, of enormous cost, were largely determined by engineering boards, and in Massachusetts and elsewhere Public Service Commissions include engineers of standing. But whatever the character of the work, its supervision by men of the type of Goethals would be a public guarantee that energy, sanity and economy would characterize the work undertaken, and we wish to voice in some measure the belief that a change in the character of Public Service Commissions so as to comprise men equipped with engineering training is essential for the best results.
>
> The practice has often obtained for the Governor of a State to ask the advice of professional bodies from which candidates can be selected to fill appointive public offices. New York is the headquarters of engineering societies comprising an aggregate membership of over 20,000.
>
> We take the liberty of urging you, first, to make the appointments of such Commissioners absolutely independent of political or other pressure, and second, to appoint men of engineering training selected by you from a list of, say, a dozen men to be submitted on your invitation by the Presidents of the five National engineering societies in coöperation with

the Presidents of the Chamber of Commerce and the Merchants' Association.

Such aggressive action by the leaders of the profession is the proper procedure if engineers are to take the prominent position with respect to public affairs that the profession as a body believes they should hold. The organizations in other professions—architects, lawyers, physicians, teachers—are active in defending the interests of their members when issues are pending in state legislatures, in Congress or before public officials. There is no reason in the world why organizations of engineers should not be similarly active in support of the interests of the profession.

Moreover, such work is actually work for the public benefit and for good government, as well as for the interest of the profession. For it is well agreed that only by putting men with some understanding of technical matters in responsible charge of engineering works, can there be any assurance of obtaining good results.

Since the above was written, Governor Sulzer has appointed two judges to membership on these Commissions. Nevertheless, the strong presentation by the Institute of Consulting Engineers was well worth making. It is only by such widely published statements as to the value of expert engineers in public service that public opinion will become educated to understand the reason for some of the serious failures of popular government.

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The Efficiency Society

Is the recently formed Efficiency Society, which held its first annual meeting in New York last week, to be classed as an engineering organization? Its principal officers are engineers and there was a sprinkling of engineers among the 80 or 100 persons in attendance at the sessions. We doubt, however, whether the organization can properly be classed as an engineering society.

In fact, so far as efficiency in relation to engineering work is concerned, it is difficult to see any reason justifying the establishment of a new society. A special section of the American Society of Mechanical Engineers has been organized devoted to the so-called science of management, which seems to fill all the necessary requirements, for the mechanical engineers at least. As for the engineers in other branches of the profession, the furor over the so called new science of management does not appear to have created any great excitement among them. It is well recognized that problems in connection with the organization and management of enterprise are a part of the engineer's job, but these problems have been with us ever since work of the civil and mining and electrical engineer began to be carried on on a large scale.

If, then, the Efficiency Society is to do a useful work, it must appeal to a much wider field than the manufacturing industries with which engineers are concerned. Indications are, in fact, that the society is endeavoring to find itself by doing this very thing. Its president is the head of a national bank, and while the majority of names on its list of directors are engineers, there are included also expert accountants, naval officers and heads of public-utility enterprises.

In the recent two-day meeting, the program may be summarized by stating that the three sessions on the first day were devoted, respectively, to the subjects of organization, management and efficiency in government work; on the second day to industrial relations and industrial hygiene.

Confining our attention to the subjects dealt with at the first two sessions, it is undoubtedly true that we need to develop a more definite science of organization and of management. It is a question, however, whether the proceedings at these sessions shed any light upon these important problems. Some of the papers appeared more likely "to obfuscate the subject," in the colored brother's words.

For example, one of the papers had the high-sounding title, "Scientific Selection and Assignment of Men in the Creation of an Organization." The young woman author of this paper laid down the admirable general principles that the work each man should be given in an industry should be the sort of work for which he is physically and mentally adapted.

As many of our readers know, the work usually known as vocational guidance consists in aiding young men and young women on leaving school to intelligent selection of an occupation, and is being developed to an important extent in connection with philanthropic and educational agencies in some of our cities.

It was soon evident, however, that it was not at all this class of work that the young woman author of the paper had in mind. Her thesis was that the employment department of an industrial concern should be "in charge of expert character analysts, who interview, select and assign men applying for positions." What she meant by "expert character analysis" was further explained in the latter portion of the paper as follows:

> He who would become proficient in human analysis must, therefore, lay tribute upon biology, anthropology, physiology, anatomy, physiological psychology, history, political economy and sociology. He must also make for himself and secure from others the results of years of patient, painstaking, widely extended scientific research into the workings of the law of cause and effect, both in nature and human nature.
>
> It is a law of nature having universal application that the size, color, form, proportion, texture, consistency, and structure of an object indicate its function and use.
>
> We determine the difference between an orange and a shotgun through comparison of size, color, form, proportion, texture, consistency and structure.
>
> The same law is applicable without modification to human beings.
>
> It is obvious that size of body fits or unfits a man for a given kind of work. If the work requires covering great distances, the short, stout man is sorely handicapped because he is not built for easy locomotion. On the other hand, much stooping or bending is difficult for the tall, rangy-built man.
>
> Blondes and brunettes do not work in the same manner. They naturally incline to different lines of work. Generally speaking, blondes are speculative and optimistic; usually mild and good natured; often inventive; like to conceive the idea or plan for someone else to work out. They naturally incline to aggressive work, such as salesmanship, advertising and promotion.
>
> Brunettes incline to conservatism rather than speculation; often serious minded; sometimes inclined to gloominess; like to work out and perfect the plan which some other mind has conceived.

The above quotation sheds some light on what the author of the paper meant by "scientific character analysis"; but a much more illuminating explanation came when, in response to a request from the audience, the young woman consented to give an exhibition of her powers of character analysis. A volunteer from the audience was brought upon the stage and the young woman, first carefully examining his hand, explained the significance of the distance between the thumb and certain of the fingers, the flexibility of the hand, the length of

the fingers, etc. Then taking a view of the subject's features she proceeded, without questioning the subject in any way, to delineate his character and explain what sort of occupation he was best adapted for. The whole exhibition was strongly reminiscent of the acute character reading which was done by traveling phrenologists a quarter of a century ago. It seems well-nigh unbelievable that it should find a place in the proceedings of a so called scientific society at the present day.

Lest it be thought that we are giving undue prominence to this feature of the Efficiency Society's program, it should be noted that it was this which attracted chief attention on the part of the public and was heralded in large headlines by the daily papers as the principal event of the session.

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Second Report of the "Joint Committee on Concrete and Reinforced Concrete"

While the "Joint Committee's" second report, presented at the recent annual meeting of the American Society of Civil Engineers, differs only in secondary detail from its first report, it has very much greater importance. To realize this it is only necessary to recall the history of the Joint Committee.

Beginning in 1904—or perhaps in reality somewhat earlier—the committee has been at work for eight and a half years; the final result is the present or second report. The first report, the product of five years' work, was the occasion of fundamental dissension within the committee, and was put forward concededly as a temporary measure, an introductory statement, a primer of the subject. Criticism apparently was expected, and it came, much of it rather sharp; the committee replied to it by stating that a mere progress report should not be expected to meet all demands. This explanation was given, for example, as a defense against the criticism that the report stated only known matters, but did not point out the doubtful or unknown matters, much less investigate them and thereby advance the art; and against the criticism that a mixture of textbook and specification matter did not satisfy the injunction "to investigate and report." Naturally the report had but a qualified standing. But the present report, coming forth as the mature product of over three years' redeliberation, is in a much different position.

We would point to the air of finality inherent in the second report. Nothing is said of tentative conclusions, of its being a progress report, of some phases having been laid aside for the present, of details still under study. No suggestion is made that the committee plans further work, or regards its duties as uncompleted, or has studies under way which will enable it to present new information in a third report. Thus the report stands forth as a real conclusion, the ultimate outcome of the committee's study of the subject. It will go before American engineers, and before our foreign brethren, as the crystallized professional opinion of this country on reinforced concrete (nominally, *concrete and* reinforced concrete; but see the contents of the report on this point).

That this makes the report a very important document indeed, will be apparent. Because it is so important, we have concluded (despite the very wide distribution which the report itself will enjoy) that an abstract of its leading features ought to be printed in ENGINEERING NEWS on the chance that a few additional persons might thus be reached. Such an abstract will be found on another page of this issue.

For study of the report, attention will be attracted by two elements: its positive statements, and the subjects omitted or ignored.

As to the first element, it will be found by the reader that the report is a sane, balanced, well-expressed, exposition of expert conclusions concerning a great number of questions arising in the use or contemplated use of reinforced concrete. Based on many long series of tests made at various places under different conditions and for different purposes, and based also on a decade's practice in reinforced-concrete construction throughout the United States, the positive part of the report takes front rank beyond all question, as a code of good practice in this field of construction.

That this second report contains only minor changes from the 1909 report, and that it agrees substantially with what has been established as good practice for some years past, proves not only that the committee did excellent work in its first period (i.e., prior to the first report), but also that reinforced-concrete engineers have been remarkably wideawake and prompt in following the experimental study of the subject and adapting their practice to the results of such study. The present report, therefore, confirms the conclusions of established practice and stamps the latter with authoritative approval.

The second element which is bound to engage the student's attention consists of the omissions of the report. No burning questions, points of doubt, subjects of criticism or suspicion, etc., appear to have representation in the report. The societies before whom the Joint Committee presents its findings will be bound to conclude that all is serene in the realm of concrete, that nothing requires further discussion or investigation. At the very least, as the committee does not report subjects of further study and, in the course of its text, makes note of practically no serious open questions, the societies must conclude that the committee did not find further questions worthy of its attention. Perhaps there will be critics of the report who will take issue with the Joint Committee on this. There may even be those who will say that the committee in reporting on removal of forms should not have omitted to point out that it advocates purely arbitrary opinion, that there is no scientific or other non-arbitrary method available, and that a test-method for deciding the time of striking centers is one of the crying needs of the art today.

Largely by the effect of omissions—for example omission of all reference to the disturbed feeling in some circles over surface disintegration and "decay" of concrete—the report has a smooth and even tone, to the degree of giving an optimistic impression. This is inevitable in part, if the report is to form a reassuring "basis for specifications." The correct modicum of doubt when referring to durability, or to effect of seawater; or a scientific disclaimer of certainty when recommending unit-stresses, etc., would not be compatible with fullest efficiency in molding the future specifications of the country. Yet there are plenty of other omitted points not under the same ban; as, the matter of a field test already

alluded to, a statement of limitations of knowledge on the agreement between structural concrete and testpieces, an opinion as to the relative homogeneity of concrete made and placed by different methods, and the like. Such omissions as the latter lead to the impression that in drawing the report the cold and plain-flavored bread of fact was buttered and spiced with a little enthusiasm, and with the wish to promote.

Now that the work of the Joint Committee is done, no further work having been reported necessary or in progress, its component committees will, we trust, request their discharge by the respective societies. No good can come of letting the subject of concrete and reinforced concrete float between heaven and earth in the custody of a committee indefinitely. The world has gravely done its share of the Joint Committee's work, by waiting nearly a decade for a final report, and it is entitled to end its waiting. Of course, this is the natural thing, and our mention of it here would be impertinent were it not for the fact that the American Society of Civil Engineers' committee (a component of the Joint Committee), which recently reported, *failed to ask for its discharge or for a specific continuance.* Is the Joint Committee to be as eternal as its subject? If so, we would advocate annual election on the same basis as governs election of officers of the societies.

LETTERS TO THE EDITOR

Purposes of Railway Signaling

Sir—I have read the articles on signaling on the C., I. & L. Ry. and the C. G. W. Ry., in your issues of Jan. 9 and 16, and have found both articles very interesting. The former article, with the summing up of the advantages of block signaling, made the better impression. In Fig. 1 of the latter article is shown a bracket post. This is not a good example of signaling, as the best practice requires the signal to be placed next to the track it governs. Placing it outside of the track and using a stub post is at best a makeshift. The diagrams in Fig. 4 of the same article show too much detail, with the result that it is difficult to trace the circuits, and the batteries and relays seem to be upside down. Some information as to the density of the traffic handled would have added to the value of the article. Signal systems accomplish two purposes: they provide "safety" and "facility," and both points should be brought out in a signal article.

H. M. Sperry,
General Railway Signal Co.

Rochester, N. Y., Jan. 29, 1913.

[The use of the bracket post in the case mentioned was due to the existence of a siding, and presumably there was not room between the siding and main track to permit of placing the signal in that position. As to the position of the batteries and relays, our readers undoubtedly recognized that these were shown only diagrammatically. We quite agree with Mr. Sperry that the operating results of signaling are fully as important and interesting as the design of signal plant.—Ed.]

Effect of Sugar on Concrete

Sir—In regard to the action of sugar and vegetable oils on concrete, mentioned in your issue of Jan. 16, 1913, the decomposing action in both cases is due to the $Ca(OH)_2$ which is split off during the hydrolysis of the silicates in the cement. In the case of oils, saponification ensues with formation of lime soap, while in the case of sugar a calcium saccharate is formed, which is soluble. The aluminates are likewise robbed of lime by sugar solutions, which result in a soft, weak, porous concrete. The remedy in each case is to reduce the amount of lime capable of being attacked. Canevazzi has reported on the successful use of puzzolana in concrete for resisting the attack of olive oil, and it is suggested that the use of tufa concrete as employed in the Los Angeles Aqueduct be employed in this case. It is further suggested that the concrete surface coming in contact with the oil be swabbed with hot paraffin and the surface then gone over with a blow torch in order to drive the paraffin into the pores. Abrasion and rough usage will not completely remove the film of paraffin and the oil will thus be kept out of contact with the $Ca(OH)_2$ in the cement.

A. J. Phillips.

U. S. Bureau of Standards,
Pittsburgh, Penn., Jan. 27, 1913.

Sir—Having spent several years as a mechanical engineer in and around sugar mills, I may be able to answer the question of F. E. P., whose letter appears in Engineering News, Jan. 16, 1913, p. 126. Your correspondent has failed to discover the probable action of sugar on concrete by laboratory tests because he has assumed that it was the sugar itself which causes the disintegration. I believe it is a fact that strong solutions of sugar and molasses do not injure concrete, and have heard it stated that the Romans mixed molasses with the other ingredients in making the concrete for their ancient structures. These strong solutions of sugar will not alter or putrefy, as is well known from the fact that sugar is used to cure perishable articles of food, but weak solutions of sugar have a very prompt manner of converting themselves into different compounds, several of which have marked acid reactions.

Around osmose presses in sugar factories, where the sugar is a weak solution, it is a common and in fact an inevitable occurrence to have the concrete floors disintegrate to a marked degree, sometimes in the short space of one campaign of 100 days and nights. Similarly, around a battery where the solution is thin a man will commonly wear out a pair of shoes in a few weeks on account of the action of the acid on the leather. I am afraid there is no way in which concrete can be made permanent in places

where weak sugar solutions come in contact with it until a good way has been discovered to make concrete acid proof.

BENJAMIN BROOKS.

1241 Holmes St.,
Kansas City, Mo., Jan. 20, 1913.

Is the Cameron Septic Tank Patent Dead or Alive?

Sir—Having noted in the press reports that the U. S. Supreme Court has not sustained our contention made in the so called Knoxville case [see ENG. NEWS, Jan. 23, 1913, p. 188], that the mere Treaty of Brussels made the 17-year term of our United States patent independent whether the foreign patents were for the same or a different invention, we believe your readers would be interested in the situation as it now stands. Our fundamental position has been, and is, that as a matter of fact none of the foreign patents cover the Septic *Process,* but merely certain *apparatus,* and it is therefore wholly inconsequent whether the foreign patents have expired or not, and our United States patent being for the *process,* is still in force and will continue so to the end of its 17-year term, Oct. 3, 1916. Thus the above mentioned decision in the Knoxville case, even if final, benefits only the one defendant in that city, the city of Knoxville, Iowa, and other infringers being in no way benefitted or relieved from liability to the end of the full 17 years' term. This fact was fully stated in an announcement made by us a year or so ago in a printed pamphlet on "The Septic Tank."

H. D. WYLLIE,
Manager, Cameron Septic Tank Co.

Monadnock Block, Chicago, Ill., Jan. 23, 1913.

Sir—On Jan. 20, the U. S. Supreme Court affirmed the decision rendered by the U. S. Circuit Court for the Southern District of Iowa in the case of Cameron Septic Tank Co. v. The City of Knoxville, and held that U. S. Patent No. 634,423 expired by limitation Nov. 8, 1909, with the expiration of a British patent granted Nov. 8, 1895, for a term of 14 years.

The claim is being generally made by the Cameron Septic Tank Co. that the effect of this decision is confined to the Knoxville case, just terminated after more than three years of litigation. It will be of interest to municipalities which have used sewage-treatment works wherein some septic action has occurred, and to the engineering world generally, to ascertain the facts in connection with this claim.

In this connection the following letter written by Wallace R. Lane, of Chicago, who argued the Knoxville case before the United States Supreme Court in behalf of the city of Knoxville, will be of interest:

Chicago, Illinois.

S. H. Crosby, Esq.,
Denver, Colorado.

Sir—Answering your letter of recent date, relative to the stipulation in the Knoxville case, I do not have before me, the statement of Geo. W. Fuller in his recent book on "Sewage Disposal," but call your attention to the stipulation made in the Knoxville case which reads:

That the allegation in the Plea to the effect that the invention patented in the United States Letters Patent, issued to Edwin [should be Donald—Ed.] Cameron, et als, on a process of and apparatus for treating sewage, No. 634,423, dated Oct. 3, 1899, set forth in said bill, had been previously patented in a foreign country by said patentee, to wit: in the United Kingdom of Great Britain, by Letters Patent, dated Nov. 8, 1895, and that the said last mentioned patent of Great Britain, expired on Nov. 8, 1909, being the expiration of the term for which it was granted, may be taken as true.

This stipulation between the parties, when taken in conjunction with the admission on the face of the Cameron et als patent No. 634,423, dated Oct. 3, 1899, that it was for certain new and useful improvements in the process of and apparatus for liquefying and purifying sewage (for which the patentees had received Letters Patent in England, No. 21,142, dated Nov. 8th, 1895, also in a large number of other countries), it seems to me, should dispose of any claim on the part of the Cameron Co. that the patent in the foreign country which has expired, was for any different or other invention than that of the United States patent, and it seems to me, such claim should not receive serious consideration by any other Court before whom the matter is presented.

In fact, in my opinion, such admissions as are made by the complainant in this case, and on the face of the patent, should be conclusive against the complainant company.

As there are numerous suits pending against various municipalities for the collection of royalties claimed under the Cameron patent, it is quite possible that the Cameron Septic Tank Co. may urge the claim referred to above in Court. It is particularly for this reason that I wish due publication to be given at this time to Mr. Lane's letter, which we [the Septic Process League—ED.] believe correctly states the legal aspects of the case.

S. H. CROSBY.

Foster Building, Denver, Colo., Jan. 27, 1913.

[In its decision of Jan. 20, 1913, the U. S. Supreme Court declared in unmistakable terms that the United States patent on the Cameron septic tank expired on Nov. 8, 1909, "with the British patent for the same invention." To sustain any claim to the contrary it would therefore seem to be necessary to get the Supreme Court to reverse its decision, either on a re-hearing of this case or by carrying some other case to the U. S. Supreme Court.—ED.]

Steamboats and Drawbridges

Sir—In the editorial on railway drawbridges in your issue of Jan. 23, you discuss why it is that the water traffic has the right-of-way instead of the railroad.

It is a common idea that, because the railroad traffic is so much greater and more important in these days of declining water commerce on rivers, the railroad should have the right-of-way. I do not believe it is at all because the water traffic was there first, but because of a very different reason which is well known to all men who have the handling and responsibility of steamboats. To arrange it any other way would endanger life and property.

Every boat has to be kept in motion to be under the control of its pilot. When he stops and floats, his boat is the sport of wind and current. In a wide stretch of the river he can wait a good while without the engines being in motion before anything begins to happen; he is comparatively like a sailor in the open sea. But as soon as a boat approaches the draw of a bridge, whether there is or is not a current, the case is different. The pilot's task becomes then the solution of delicate problems of current, wind, momentum, steering power and velocity through the water. All pilots know that they are perpetually combining forces into a resultant that means the wrecking or the safety of the boat.

When near or in the draw opening of a bridge, especially when the current is swift and perhaps a side wind

springs up, you will see the knees of the older pilots knocking together if on a large boat—for they have a keen appreciation of what is going to happen if the calculations are out or if the bridge fails to open as it should before they get too close to be able to save the boat by backing away.

There are bridges on the upper Mississippi River where the current at high water is as much as eight miles an hour. Now few boats going ahead at full speed upstream make much more than that; astern they go much slower and are also much less under command.

Imagine a big boat approaching a draw such as that from upstream and a railroad man undertaking to hold him up with a train; and consider the feelings of the pilot and such of the passengers as have any knowledge of the dangers of steamboating. The train can always stop. It cannot be blown from its course. It undergoes no danger by stopping and there are no currents to bear it down on a bridge and tear its side out against pier or abutment.

The steamboat, then, has the right-of-way because it is, compared with the train, much less under control, and because this is the only way to protect the passengers and crew.

M. MEIGS.

U. S. Civil Engineer, Keokuk, Iowa,
Jan. 26, 1913.

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The San Francisco Water-Supply Hearing before the Secretary of the Interior

Sir—In your issue of Dec. 26, 1912, you print in connection with a review of John R. Freeman's report on the San Francisco water-supply a "Summary of Hearing before Secretary Fisher on the Hetch Hetchy Water-Supply," written for you by Mr. Freeman.

While having full regard for the large experience and great engineering ability of Mr. Freeman as manifested in that part of his report which resulted from his own investigations, I do not think it fair that his summary should be considered as complete, inasmuch as it does not mention many important statements brought out by questions of the Secretary of the Interior and others.

The Secretary secured the admission from Mr. Freeman that the figuring involved in his report on the Hetch Hetchy supply was not on a parity with the figuring involved in his report on the alternative sources of water-supply. It was made known, as the inquiry proceeded, that in relation to these alternative supplies Mr. Freeman had been content to use the figures of Messrs. Manson and Grunsky, while he had discarded the figuring of these engineers and had, indeed, shown a lack of confidence in their statements concerning the Hetch Hetchy project. It was also shown that a different unit basis had been used in figuring the costs for the Hetch Hetchy project and in connection with the one alternative source upon which engineering investigation of recognized adequacy was used—that of the Sacramento River, reported upon by Allen Hazen.

The result of these disclosures was to have the Secretary order Mr. Freeman to at once have all the figures put on a parity, so that fair comparisons might be made.

Nor has Mr. Freeman presented the fact that while, as he has said, the various engineers called by the City of San Francisco, with one exception, agreed that the Hetch Hetchy provided an admirable natural source for a municipal water-supply, one of them, Prof. Whipple, said very definitely that "the time is coming when all surface water-supplies will be filtered," and another, Mr. Hazen, showed that the claim as to the comparative softness of the Hetch Hetchy supply was not really important.

Surgeon-General Rupert Blue—who, it will be remembered, was opposed and denounced by the San Francisco officials when he was conducting his memorable campaign against bubonic plague in that city—disagreed with Mr. Freeman and his associates as to the propriety of sanitary reliance upon the purity of any surface source of water-supply when he definitely announced his preference for the filtered water of the Sacramento River rather than the unfiltered Hetch Hetchy water. He was not willing to "skate on thin ice" with the lives of a great urban population, in respect of its water-supply.

Secretary Fisher's searching questions as to the use of the power which might be developed from the impounded Hetch Hetchy water developed an interesting disposition on the part of Mr. Freeman, for he took the stand, as shown by my notes made at the moment, that it would not be wise for the City of San Francisco to develop this power and sell it to its citizens at cost, or to use it in promoting industrial advance. He believed that there should be a profit charged, which he said might be used in "municipal embellishment," and his given reason was that there should be no municipal interference with private corporate enterprises selling electric power.

I wish Mr. Freeman, for whose purely engineering conclusions I have great respect, had kept closer to the facts in his criticism of the opponents, whom he groups under the head of "nature lovers," and who he says "claimed that none of this land, dedicated to the whole of the American people, should be diverted so long as another source was available, regardless of cost." He does me the credit of saying that I did not go quite so far as to agree that the city ought to be "compelled to distill water from the Pacific Ocean," rather than have the Hetch Hetchy supply.

Now the truth is that I said many times in Mr. Freeman's hearing, and to him separately, and have before and since repeatedly put the statement in writing, that I believe, and the American Civic Association stands for the position, that San Francisco should have the Hetch Hetchy Valley, or any other property of the Federal Government, for the purposes of a municipal water-supply, if there is available to her no other practicable source of a water-supply. And this is the position that has been steadfastly maintained by a large majority of those who are questioning San Francisco's attitude in this matter.

The City of San Francisco has been at this proposition for eleven years. All its engineering findings up to the time when Mr. Freeman took charge are by him discarded as valueless, save as they affect the alternative supplies, his consideration of which was seemingly for the purpose of discovering not their practicability, but their impracticability. After all this lapse of time, after the expenditure of more than a million dollars, and after the use by the City of San Francisco of Mr. Freeman's genius on the proposition for about two years, the fact still remains, as brought out at the hearing in question, that no adequate and impartial investigation of all the available

sources of a water-supply for the city of San Francisco has yet been made.

The American Civic Association desires nothing but the greatest good to the greatest number in this matter. It will cheerfully second San Francisco's desires when and if those desires are fairly and honestly shown to be in accord with this principle. That the people of San Francisco have a great doubt as to the practicability of the scheme was rather conclusively shown in the election of Dec. 10, when the charter amendments relating to the progress of the Hetch Hetchy project were rejected by majorities approximating three to one.

J. HORACE MCFARLAND.
President American Civic Association.

Harrisburg, Penn., Jan. 13, 1913.

[In accordance with our custom in such cases, we submitted a proof of Mr. McFarland's letter to Mr. Freeman. The reply of the latter is printed below.—ED.]

Sir—I submit the following reply to Mr. McFarland's letter of Jan. 13:

Obviously, all of the interesting statements made in the five-day hearing could not be reported in the space given to this subject in your issue of Dec. 26. For example, early in the hearing I felt compelled to enter a protest against Mr. McFarland's unjust accusations and wrong statements regarding my presentation of the city's case. Apparently these gentlemen do not understand that an engineer may value and guard his reputation for truth and fairmindedness just as highly as fair words concerning his professional skill.

In the foregoing letter from Mr. McFarland the same spirit of unfair criticism is found in such phrases as "the secretary secured the admission," "result of these disclosures," and the remarks on my lack of confidence in the statements of the two former city engineers.

The facts regarding all of these matters were set forth with the utmost plainness in the reports as originally published. The one important difference was that I used the same price per day for common labor that prevails today on most similar large construction work in California and that which was actually paid on the Los Angeles aqueduct, while certain of the other engineers used the price paid to city laborers in San Francisco, in accordance with an ordinance which will probably be changed. Each engineer's estimate was published over his own signature and with the plainest kind of statement relative to his own unit costs. In my own reports these matters, as any fair-minded man familiar with engineering reports on public works must admit, were set forth with a minuteness of detail that is seldom published.

Mr. McFarland and his associate critics who were not engineers, and who were apparently unfamiliar with engineering estimates, in their comparison of unit costs failed to note that in the alternative projects contractor's profit and general overhead expenses and loading for administration were all lumped into the cost per yard or per foot, whereas in my estimates the "field cost" was first separately reckoned and a careful estimate made in detail for the "loading" or overhead costs, amounting in general to 40% and in some cases to much more.

The supplementary statements filed with the Secretary of the Interior since these hearings, in general, make the difference of cost in favor of the Hetch Hetchy project substantially as great as was originally claimed.

Mr. McFarland ingeniously states that his association stands for the position that San Francisco should have the Hetch Hetchy if there is available no other practicable source of water-supply, but evidently fails to consider that an excess of from twenty to thirty million dollars in the cost of alternative projects, as compared with the cost of the Hetch Hetchy, makes them practically unavailable.

Your space is too valuable to be taken up with such petty matters as these, or with going into details about the biased character of the letter ballot sent out by the Sierra Club regarding the Hetch Hetchy project.

JOHN R. FREEMAN.

Providence, R. I., Feb. 1, 1913.

[In the Hetch Hetchy controversy the city of San Francisco has had to contend with (1) the Spring Valley Water Co., (2) the Turlock and Modesto Irrigation Districts, (3) proponents of rival sources of water-supply, and (4) various associations and individuals who for convenience and sometimes with unmerited scorn have been classed together as "nature lovers," because they are united in opposing what they believe would be the destruction of a beautiful mountain meadow in one of our national parks—but what San Francisco claims would be the mere conversion of a part of a mountain meadow into a mountain lake, and would make the section accessible to the many instead of to the few.

In this controversy, the city, the water company, the irrigation districts and the would-be sellers of rival sources of water-supply have all been represented by engineers, including a number of the foremost hydraulic engineers of the United States. The subject has been vast and complex and involves millions of dollars of rival private commercial interests, as well as the future health and prosperity of San Francisco.

Under such circumstances it is not strange that the "nature lovers," being human, have shown a strong tendency to accept without question the engineering and other claims made by those whose commercial interests ally them against San Francisco and the Hetch Hetchy project, and an equally strong tendency to discredit the testimony put forward in behalf of San Francisco.

Mr. Freeman has touched upon a number of the points raised by Mr. McFarland. He has not, however, called attention to the fact that the San Francisco charter amendments, mentioned in the last paragraph of Mr. McFarland's letter, did not deal directly with the merits of the Hetch Hetchy project, but with such questions as authorizing a separate water-supply board and financing the project outside of the 15% debt limit. Moreover, these questions were tied in with other ones, such as creating a city public-utility commission. Finally, the civic associations of San Francisco united in advising that these and a number of other of the 37 charter amendments voted upon on Dec. 10 be defeated, but none of the arguments which they advanced contained a word against the Hetch Hetchy project.—ED.]

The Machinery Building of the Panama-Pacific Exposition, at San Francisco, Calif., which was put under construction on Jan. 1, 1913, will be the largest frame building in the world, according to its architect, Clarence Ward. It will require "8,000,000 cu.ft. of lumber," it is stated.

How to Repair the Nashville Masonry Reservoir

SYNOPSIS—Rudolph Hering concludes that further leaks into the shaly foundation of the Nashville reservoir, 150 ft. of the wall of which slid out on Nov. 4, may be prevented and the East Basin made safe at a cost of $95,115. Besides replacing the breached wall, it is recommended that the East Basin be given a cement-gun coat of mortar, and the floor of both basins be waterproofed by asphalt felt, protected by concrete, and that a waterproofed wedge be built at the junction of the wall and bottom, and a drain placed beneath this wedge. This done, similar reinforcement of the West Basin is recommended.

The failure of a portion of the wall of one basin of a water-works reservoir at Nashville, Tenn., on Nov. 4, 1912, was described and illustrated in our issue of Nov. 14, 1912, pp. 922-4.* A report reviewing the failure briefly and recommending measures for repair was submitted to the Board of Public Works of Nashville, under date of Jan. 15, 1913, by Rudolph Hering, of Hering & Gregory, New York City.

On Nov. 24, 1912, Mr. Hering visited Nashville upon invitation of George Reyer, superintendent of water-works. A few facts and opinions based on Mr. Hering's inspection of the damaged reservoir will be abstracted from the report to supplement the account of the failure published in our issue of Nov. 14, 1912. We shall then print the substance of that portion of the report which deals with proposed repairs to the reservoir.

The general design of the reservoir and its dimensions, the cross-section of the wall which failed and of the wall designed to replace the breach, and the details of the proposed lining, wedge and drains, are all shown by the line drawings reproduced herewith. The capacity of the reservoir was 50,000,000 gal., divided equally between the two basins. The water line was about 31 ft. above the bottom of the reservoir.

The reservoir was located on the top of a domeshaped hill. The outer parts of the walls were built of limestone, excavated from the site and the inner part was composed of concrete. Louisville natural cement was used throughout. The walls were founded upon "much jointed limestone beds, between many of which are thin layers of shale" (A. H. Purdue, state geologist of Tennessee). At many points, Mr. Hering says, the foundation course of the walls starts not more than a foot beneath the natural surface.

About 150 ft. of the outer wall of the reservoir was pushed outward (see plan). Mr. Hering describes and comments on the break as follows:

> An examination of the rock beneath showed that the original shale had been altered to clay at a number of points. It was further found that the failure was apparently due to a slip on a layer of the altered shale, which was about 4 ft. below the top of the rock upon which the wall had been placed. This was evident because about 4 ft. of bed rock adhered to the base and was moved out with it. That there had been a slip was shown also by the fact that the surface of parts of this clay layer was quite smooth, showing that the superimposed material had slipped upon it.
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> The material of the wall exposed at the breaks was fairly good. No disintegration appeared. The cement mortar was generally hard, though here and there quite porous. In some cases it was stronger than the limestone, the latter breaking rather than the cement pulling off. In other cases it was rather soft and could be crumbled with the fingers. Some of the limestone in the wall is of excellent quality; some is poor, laminated and weathered. At some points it was evident that percolation through the wall had been going on for some time. It can be said that generally the masonry and core wall were good and built as average work was usually done at the time of its building, and were in a good condition.
>
> The foundation courses were not found to be so well built. Mortar here and there was more scarce and the masonry was not as compact as in the upper parts. In other places, there were indications that mortar had been used plentifully, indicated by the fact that some of the foundation stones adhered to the rock layers better than the rock layers adhered to each other, and indicated by the fact that portions of the original rock moved out with the wall. The shale and clay layers in the original rock under the wall were quite frequent and varied from ⅛ in. to 5 or 6 in. in thickness.
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> Before the break occurred, water at the location of the subsequent break ran down freely on the front of the wall, into the natural ground beneath, and into and below the foundation, where it evidently soaked into the soft layers of clay or shale, which had been transformed into clay. This percolation was probably the final cause for removing the frictional resistance of those transformed layers. The percolating water had thoroughly saturated and softened them. They had become slippery like a lubricated surface.
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> The first rupture no doubt occurred at the central break, where the wall had been pushed out furthest, and the other breaks, one on each side, were consequential.
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> When the rupture occurred and the wall was pushed out, together with several feet of bed rock holding to it, the mass pushed out the rock layers in front sufficiently to cause a fold in the layers, which was quite apparent at the inspection. A thick clay layer located about 4 ft. below the bottom of the wall is probably the one which caused the initial slip.
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> There seems to be some doubt as to whether the entire floor of the reservoir was covered with Louisville cement concrete and if so, what the depth of concrete was. At the break it is evident that such a floor existed, but the depth may vary. At the places observed it was about 12 in. thick.

The walls have stood 23 years without significant, if any, settlement. They have a sufficient cross-section to store water to a depth of 31 ft. and are fairly well built, but they leak at numerous places and this leakage flows down their outer side and is liable to cause softening of the clay beneath the foundations and thus give rise to further slips. "There is no evidence of percolation through the bottom of the reservoir," Mr. Hering states, "but as such percolation is not improbable it should be assumed that it occurs."

After considering all the governing conditions, Mr. Hering concludes that the reservoir can be made safe by preventing further leakage into the natural foundation. This can be done, after closing the breach, by lining the inner walls and the bottom, and placing a waterproofed concrete wedge and a drain at the junction of the two. The nature of this work is explained as follows (see also the detailed drawings):

> (1) In order to secure watertightness of the present walls they should receive a lining of cement mortar over the entire inside surface of the reservoir, applied by what is known as the cement gun. Wet cement mortar of a proper mixture is shot against the surface of the wall, after this has been thoroughly cleaned, first by thorough washing and scraping, then by sand blasting, which will make the adhesion of the cement complete. The density of this gun cement is very great, and it penetrates into all open crevices and covers every fine crack. An average thickness of about an inch is more than enough to make a wall entirely watertight. Details of application are given in the appended specifications.
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> (2) In order to make the floor of the reservoir watertight it should receive, after thorough cleaning, a layer of 5-ply asphalt felt [see detail]. Upon this layer, which secures the watertightness of the floor, there should be placed, to protect it, preserve it and keep it in place, a 4-in. layer of

*See also Eng. News, June 15, 1888, p. 484, for description of original construction, written by H. de B. Parsons.

portland cement concrete. This layer should be put down in 16-ft. squares, in a manner and jointed as indicated in the specifications and plans.

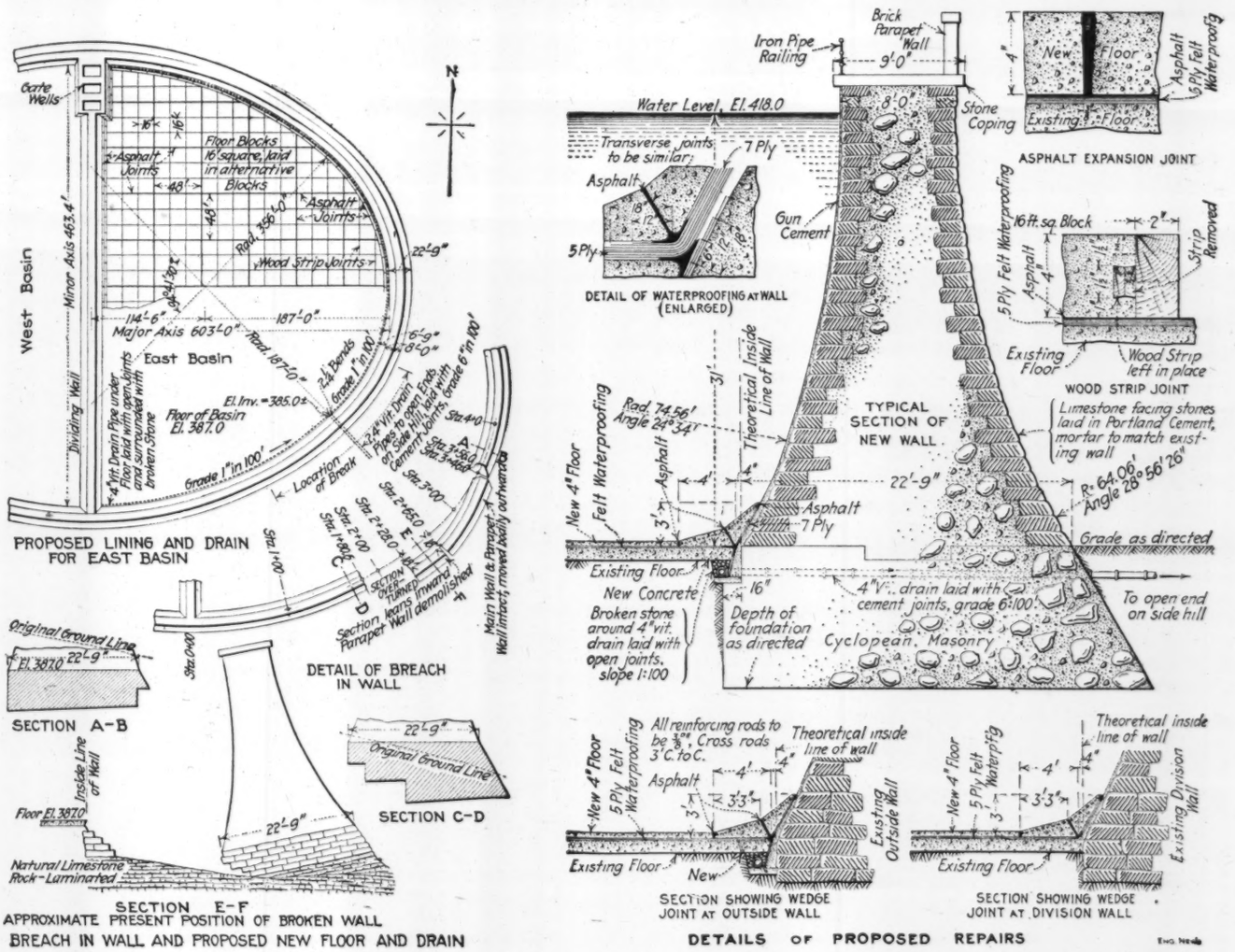
Dividing the concrete floor into 16-ft. squares has the purpose not only of obtaining better joints between the separate applications, but also of providing facilities for an expansion and contraction of the concrete, due chiefly to temperature changes.

(3) Having watertight walls and flooring, it becomes necessary to provide also for a thorough watertightness of the joint between the wall and the floor. This joint is liable to change in width, because there is a possibility of the walls settling slightly in the future, due to the altering of the shale layers. A settling, even the most insignificant, will produce a crack, and this would allow water under 31 ft. of head to penetrate and enter the clay layers. The width of the joint is also affected by contraction of the concrete of the floor. Provision must, therefore, be made to allow for these probable vertical and horizontal movements in a manner to obviate the formation of any crack with consequent leakage.

This condition can be secured by placing at the inside foot of the wall a large wedge of concrete, the two sides being at an angle of 60 degrees from the horizontal, and both sides resting upon layers of waterproof materials, one upon the wall and the other upon the floor, so that any vertical movement of the wall or horizontal movement of the floor will correspondingly move the wedge upon the asphalt surfaces, and, therefore, avoid any tendency to cause a crack, either in the floor or at the wall [see detail.] The water pressure in the reservoir would continually tend to press the wedge against the adjoining surfaces, and not prevent its rising in case there is an expansion of the floor.

(4) Below the wedge it is proposed to lay a 4-in. drain pipe [see detail] on a slight grade around the entire inside of the reservoir, discharging with two outlets outside of the wall, so as to give assurance, either that the leakage within the reservoir, if there is any, is insignificant, or and mainly, that any leakage water will neither soak down into the clay strata beneath the walls and lubricate them, nor exercise any upward pressure within the wall, to reduce its weight, and, therefore, its stability.

This drain pipe will be a safe indicator of the watertightness of the reservoir. If either or both of the pipes do not discharge any water outside of the reservoir, its watertightness and the entire safety of the walls is demonstrated. If either or both show a flow of a constant but small amount of water, it indicates a small leakage, which should be observed and gaged to ascertain whether it is constant or increases. A constant small leakage does not indicate any danger. If such a leakage increases continuously and becomes large, the reservoir in which it occurs should be emptied, and the location and cause ascertained and remedied. Had such means existed in the present reservoir, an examination and repairs could have been made in ample time and the break obviated. After the [East Basin has been repaired and reinforced] the West Basin should then be [reinforced] in the same manner, so as to prevent any slippage and break occurring in it, as [its] wall is also built upon the shale and clay strata, which underlie the entire reservoir, and which might in the future also cause a slippage as in the East Basin. Horizontal leakage now occurs to some extent through the wall of the West Basin below the present water level.


Gate Wells
West Basin
Minor Axis 463.4'
Dividing Wall
Asphalt Joints
Floor Blocks 16' square, laid in alternative Blocks
Wood Strip Joints
Rad. 356'-0"
Major Axis 603'-0"
Rad. 187'-0"
East Basin
Floor of Basin El. 387.0
El. Inv. = 385.0±
Grade 1" in 100'
Location of Break
PROPOSED LINING AND DRAIN FOR EAST BASIN
Original Ground Line
El. 387.0
SECTION A-B
Inside Line of Wall
Floor El. 387.0
Natural Limestone Rock - Laminated
SECTION E-F
DETAIL OF BREACH IN WALL
SECTION C-D
APPROXIMATE PRESENT POSITION OF BROKEN WALL
BREACH IN WALL AND PROPOSED NEW FLOOR AND DRAIN
Iron Pipe Railing
Brick Parapet Wall
Water Level, El. 418.0
Stone Coping
Transverse joints to be similar
7 Ply
Asphalt
5 Ply
Gun Cement
DETAIL OF WATERPROOFING AT WALL (ENLARGED)
New Floor
Existing Floor
Asphalt 5 Ply Felt Waterproofing
ASPHALT EXPANSION JOINT
16 ft. sq. Block
Strip Removed
5 Ply Felt Waterproofing
Wood Strip left in place
WOOD STRIP JOINT
TYPICAL SECTION OF NEW WALL
Theoretical Inside Line of Wall
Rad. 74.56'
Angle 24° 34'
New 4" Floor
Felt Waterproofing
Limestone facing stones laid in Portland Cement mortar to match existing wall
R = 64.06'
Angle 28° 56' 26"
Grade as directed
New Concrete
Broken stone around 4" vit. drain laid with open joints, slope 1:100
Depth of foundation as directed
Cyclopean Masonry
4" V. drain laid with cement joints, grade 6:100
To open end on side hill
All reinforcing rods to be ⅜", Cross rods 3' C. to C.
Existing Outside Wall
SECTION SHOWING WEDGE JOINT AT OUTSIDE WALL
Existing Division Wall
SECTION SHOWING WEDGE JOINT AT DIVISION WALL
DETAILS OF PROPOSED REPAIRS

PLAN AND DETAILS OF BREAK IN MASONRY RESERVOIR, NASHVILLE, TENN., AND OF PROPOSED REPAIRS AND IMPROVEMENTS

ESTIMATES OF COST

The following is an approximate estimate of the cost for making the repairs of the East Basin of the reservoir and of

giving it such additional treatment that it will in future be watertight:

ESTIMATED COST OF REPAIRING THE EAST BASIN AND OF RENDERING IT WATERTIGHT

Item	Unit	Quantity	Price	Amount
Earth excavation	cu.yd.	10,500	$0.40	$4,200
Rock and masonry excavation	cu.yd.	4,000	2.00	8,000
Concrete, Class A	cu.yd.	1,800	6.50	11,700
Concrete, Class B	cu.yd.	2,800	7.50	21,000
Concrete, Class C	cu.yd.	1,600	8.00	12,800
Brick parapet wall	lin.ft.	180	4.00	720
4-in. vitrified pipe	lin.ft.	1,000	0.20	200
Broken stone or gravel	cu.yd.	40	2.00	80
Cleaning walls	sq.yd.	5,100	0.10	510
Sand blasting of walls	sq.yd.	5,100	0.50	2,550
Plastering with cement mortar	cu.ft.	11,000	0.50	5,500
Asphalt waterproofing	lb.	30,000	0.03	900
Asphalt felt waterproofing	sq.yd.	12,350	1.00	12,350
Iron pipe railing	lin.ft.	200	1.00	200
Steel reinforcing rods	lb.	6,500	0.04	260
				$80,970
Engineering and contingencies, 15%				12,145
				$93,115

Concrete Class A includes the masonry in the foundation of the reservoir wall below elev. 387.0. Its composition is 1:3:5.

Concrete Class B includes the masonry in the reservoir wall above elev. 387.0. Its proportions are 1:2:4.

Concrete Class C includes the masonry in the floor of the reservoir, the wedge blocks, the foundation under the vitrified drain, and the replacing of the existing floor where removed. Its composition is 1:3:5.

Sand and broken stone are used in each of the three classes. Large stones may be added to Class A and Class B concrete, if so desired, in which case it shall constitute cyclopean masonry.

The original cost of the reservoir, as given in "The Manual of American Water-Works" for 1889-90, was $385,000, or about $7700 per 1,000,000 gal.

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The Annual Meeting of the Indiana Engineering Society

The 33d annual convention was held in Indianapolis, Ind., Jan. 23-25, inclusive. This convention was a notable one in the interest shown and in the breadth and scope of the subjects discussed, which included not only engineering problems, but such public and social problems as the housing laws, greater safety in steam and electric-railway operation and workingmen's compensation laws.

On Thursday, Jan. 23, there was a special meeting of county surveyors. A temporary organization was effected, the purpose of which will be to obtain, by legislative action, uniformity in salaries paid to county surveyors, and to provide for a percentage fee to be paid them on the contract prices of roads and bridges built under their supervision. About 25 county surveyors were present; J. B. Short, of Fountain County, was elected president of the organization.

The first regular session of the society took place Thursday afternoon. In his presidential address DeWitt V. Moore, of Indianapolis, urged the society to use its influence for the construction of good roads; he also recommended that the society give attention and consideration to such problems as accident prevention, occupational diseases, housing reform, etc. He stated that the assumption of these burdens was a duty which engineers owe to their fellow men, who were, perhaps, in carrying out their daily labor, accomplishing the fulfillment of engineers' plans and designs.

HOUSING REFORM—This subject was taken up in a paper read by Wilson B. Parker, an architect, of Indianapolis, entitled "The Housing Law in America." He emphasized the need of proper housing legislation to protect the health and morals of the poor in our great cities. The necessity of good fire escapes, sewerage, light and air shafts and the limitation of the space to be occupied on each lot by buildings were considered in detail. Mr. Parker's paper was illustrated by lantern slides showing disadvantageous conditions in Kansas City and Philadelphia.

SAFETY IN RAILWAY OPERATION—Prof. H. O. Garman, of Purdue University, chairman of the committee on steam railways, reported that but 22% of the railways of Indiana had been equipped with block signals before the state railway commission began to require their adoption; when the work recently ordered by the commission is completed there will be 64% of the railway mileage thus safeguarded. E. G. Stradling, of Lafayette, read a paper on the "New Automatic Block Signals of the Monon (Chicago, Indianapolis & Louisville) Ry." The operation of electric railways was treated by Alex Shane, of Columbus, Ohio, in a paper entitled "A Movement to Secure Greater Safety in the Operation of Electric Railways."

MECHANICAL ENGINEERING—Friday morning's session was devoted to the consideration of mechanical engineering subjects. The mechanical equipment of the Severin Hotel, Indianapolis, was described by O. N. Mueller, of that city. J. S. Stoudt read a paper on "Automobile Starters.' The manufacturers' side of the problem of industrial accidents and workmen's compensation was discussed in a paper read by M. W. Mix, of the Dodge Manufacturing Co., Mishawaka, Ind. A paper on workmen's compensation was also read, by the president, Mr. Moore. "The Value of an Apprenticeship Course to the Manufacturer and the Apprentice," was the title of a paper read by H. S. Dickerson, of Lafayette, Ind. Prof. L. W. Wallace, of Purdue University, read a paper on the "Relative Resistance of New and Worn Car-Wheel Treads."

SANITARY ENGINEERING—H. W. Klausmann, City Engineer of Indianapolis, presided over the Friday afternoon session. Papers were read, "A Sanitary Survey of the Wabash River," by J. A. Craven, of Indianapolis; "The Use of Hypochlorite of Lime in Water Purification," by Prof. C. N. Hilliard, of Purdue University; "The Hydraulic Laboratory at Purdue University," by Prof. R. L. Sackett; "The Indianapolis Experimental Sewage Disposal Plant, and Notes on Sewage Disposal in Other Cities of the United States," by H. W. Klausmann; "The Julietta, Ind., Sewage Disposal Plant," by Charles Brossmann, of Indianapolis," and "Observations and Recommendations as to the Methods of Conducting Drainage Works," by E. E. Watts, of Princeton, Ind.

ROAD CONSTRUCTION—Friday evening the members of the society and their guests enjoyed what was termed a "good-roads smoker," at which there were informal discussions of road building by James T. Voshell, Highway Engineer, U. S. Office of Public Roads, Capt. C. S. Tarleton, U. S. A. (retired), who spoke on Philippine roads librarian, of Indianapolis.

CONCRETE—The session on Saturday morning was presided over by D. B. Luten, of Indianapolis. Papers were read entitled, "Indiana's Concrete Aggregates," by C. D. Franks, of Chicago, Ill.; "A Study of the Artistic Treatment of Concrete Bridges," by John Mueller, of New Castle, Ind.; "The Tennessee Ave. Bridge, Ft. Wayne, Ind.," by A. W. Grosvenor, Ind.; "An Ordinance for Reinforced-Concrete Building Construction," by Prof. W. K. Hatt, of Purdue University.

RESOLUTIONS—The society adopted resolutions favoring the creation of a state highway commission and state aid for highway construction; a resolution was also adopted favoring a $10 per diem recompense for engineers on public work, or remuneration based on a percentage of the contract price.

OFFICERS—The officers for 1913 are: President, H. O. Garman, Associate Professor of Civil Engineering, Purdue University, and Secretary, Charles Brossmann, 43 Union Trust Building, Indianapolis, Ind. The next meeting is to be held at Purdue University, Lafayette, Ind.

Mid-Year Conference of the American Electric Railway Association

Since the development of the mid-year conference of the American Electric Railway Association at headquarters in New York City, these meetings have developed to an importance barely second to that of the annual convention in the fall. In fact, so far as pushing the work of the Association goes, it is possible that more is actually accomplished because of the fewer general sessions and greater opportunity seized for committee work. Some 47 committee meetings were scheduled for the four days of the last conference, Jan. 29 to Feb. 1. At these committee meetings the work done since appointment at the close of the last fall convention was reviewed, or in some cases the organization was perfected so that scheduled work could be taken up. The morning and afternoon of the third day of the conference, Jan. 31, were given over to general sessions. At a banquet held with the allied manufacturers' association on Jan. 31, there were about 900 present.

At the first of the general sessions of this conference the report of the Committee on Proper Basis for Rates and Fares, as presented before the last convention, was discussed. In re-presenting the report, F. R. Ford, of New York City, supplemented it with a recommendation that the work of such a committee be continued on a permanent basis (1) with salaried officials or an organized department for collecting statistics on fares, and for coöperation with member companies, (2) for encouraging and sustaining desirable public relations along this line, (3) for the formulation of standard accounts and forms of studying railway costs and fares. These recommendations were later approved and referred to the executive committee of the Association.

C. N. Duffy, of Milwaukee, a member of the same committee, presented a paper on "The Effect of Load Factor on Cost of Electric Railway Passenger Service." In this he followed a method developed by the Wisconsin Railroad Commission in studying rates of fares. "Load factor" was defined as the number of passengers carried divided by the seating capacity of cars in operation. For studying the effect of load factor on total cost, the costs were divided into (1) expenses which exist without traffic, (2) expenses proportional to car traffic, (3) expenses proportional to number of passengers. The Wisconsin Commission has found, he reported, that the first of these comprised 82.4% of the total cost of service, and that the total cost per passenger decreased as the load factor increased. For a flat fare of 5c. per passenger, the maximum possible length of haul under ordinary conditions increased from 1.36 miles at 10% load factor to 4 miles at 40%, and 8.80 miles at 100%. The author held that low rates of fare during rush hours could not be justified, but that the cost of service and rate of fare could be reduced and length of haul could be increased, were it practicable to increase the load factor by spreading the traffic over a greater number of hours per day. Where operating expenses, depreciation, extraordinary contingencies, factors, investments, etc., were identical between two electric-railway systems, the reasonable length of haul was dependent upon load-factor conditions which had to be studied in determining fares.

W. J. Clarke, of the Traction Department, General Electric Co., Schenectady, N. Y., presented a paper filled with data on British tramways. He outlined the financial advantages which these municipal projects enjoyed in the matter of low, but rising, rates of interest, low wages ($6 to $8.75 for motormen and conductors after five years' service), etc. In spite of these advantages he found but 2637 miles of tramway lines—only as much as in New York or Pennsylvania. He held that both American and British roads over-estimated their net earnings and failed to provide adequate funds for obsolescence, made too long life allotments of equipment and too low rates of fare.

British tramways were also discussed by A. K. Baylor, of New York City, who described the rise of the municipal tramway in England as being due simultaneously to (1) expiration of the horse-car-line franchises in England, (2) the success of an electric-lighting department in Birmingham, (3) expansion of the idea of municipal control, and (4) tremendous development of electric traction in the United States. The low rates of fare he explained as being due to the prevalent small unit of money (penny), there being no coin in general circulation there corresponding to the American nickel.

R. B. Stearns, of Milwaukee, presented a paper on the use of the zone system of collecting fares. He had come to believe thoroughly in this arrangement and advocated collecting five cents for the zone originating traffic and an additional two cents for each zone outside the first district. He believed that a two-cent coin would be found admirably suited to such a system and advocated its issue from the mints.

C. S. Sergeant, of Boston, in describing the upgrowth of the electric-railway system of that metropolitan district, stated that in 1888 five or six horse railways were consolidated, the combined capital being 2.7 times the gross earnings. The average number of passengers per one-half round trip was then 22½, with an average length of passenger ride of 3.5 miles, and an average length of car trip of 4.8 miles. After electric traction was installed, in 1892, the number of passengers per half trip rose to 28, and in 1898 it still further increased to 29½. But after true rapid-transit facilities were provided, the average number of passengers (in 1903) dropped to 23, and gradually rose to 25 for one-half round trip in 1912. The drop was due to more trips and longer extensions of line. The permanent investment at the present time was six times the gross earnings, due to the cost of rapid-transit facilities. All these he mentioned as illustrating some of the points in the committee's report and in Mr. Duffy's discussion.

The second session on the same day was given up to a discussion of the recent report of the U. S. Railway Securities Commission. The first speaker was D. W. War-

ren, general counsel of the Bay State Street Railway Co., Boston, Mass. He noted that while electric railways were largely not interstate organizations, yet the remarks of the commission applied very well to their financial operations. He described the effects of the drastic Massachusetts anti-stock-watering statutes, which were annoying to an honest concern as well as one dishonest. He stated that it had been very difficult to obtain capital for transportation companies since the law had prohibited the issuing of stocks below market value. He cited, in this connection, the difficulties experienced by the Bell Telephone Co. and the Boston & Albany R.R. Lack of provision for bond discounts and working capital also came in for criticism.

The paper "Mistaken Popular Notions Concerning Public Service Corporations" was read by Frederick Strauss, of J. & W. Seligman & Co., of New York City, and a member of the Securities Commission. His contentions were that the so called "special privileges" were not privileges at all, but necessary rights without which these utilities could not be developed—such things as rights of eminent domain, perpetuity of life, limited liability of stockholders, and the right to act by majority vote. The element of monopoly he held was one of fact rather than of law and that while the law permitted or invited competition, public interest and economic conditions usually dictated a monopoly. He held also that the public was confusing "reasonable return" with mere "abstention from confiscation," which would not suffice to bring out capital needed for extensions, improvements and progress in services.

In discussing "watered stock," he claimed this was to be found in one form or another in the older countries. He commented on the ability of the great banks in Europe with enormous capital and surplus to carry on new projects until the earnings have reached the desired point where shares were sold to investors at a high premium. In this way, he said, the promoters reaped a large profit by sustaining the enterprises during infancy and finally the capital of the bank was freed for new ventures. In this country, the banks could not do this with their funds and the necessity for setting capital free at the earliest possible moment was the cause of the issue of so called "watered stock" and the advocacy of shares without par value.

O. T. Crosby, of Wilmington, Del., maintained that it was highly desirable that the academic side of the business should be studied, including just such questions as were treated in Mr. Strauss' paper. He believed that self-education and education of the public were necessary before the electric-railway industry was put on a proper financial basis. He held that proper representatives of the association should appear before every legislative body in the country, which was considering questions involving these utilities, and that they should plead general economic principles. He claimed that there were no precedents to govern the financing regulations which were to be imposed on these utilities; the industry was so bound that there was no basis for stating, for instance, whether 6%, say, was confiscatory or not.

The discussion for a while turned on physical valuation, Prof. A. S. Ritchy, of Worcester, Mass., and C. G. Young, of New York City, maintaining that the term itself should have been defined when used, as by the Railroad Securities Commission, and that in making any valuation the use should be fundamentally considered. Mr. Young held that valuations were often of questionable service anyway. Prof. Ritchy deprecated the tendency to allow a return only on depreciated value of a property which had been in service a number of years.

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Specifications and Design Standards for Reinforced-Concrete Structures*

By Ernest McCullough†

Engineers should do more than they are at present doing, to post owners and architects on proper design and to expose faulty methods in reinforced-concrete work. In the first place, the average architect and owner rely too much upon the salesmen for steel companies for plans of reinforced-concrete buildings. They think the construction practically standardized, so that when competitive designs are submitted, all designers work on the same basis. A common requirement in specifications is that the building shall be designed to carry certain floor loads with a factor-of-safety of four. The owner may think that under this clause a floor to carry safely 100 lb. per sq.ft. will be apt to break under 400 lb. Many architects say that this means four times the dead-load plus the live-load the floor is expected to carry constantly. The engineer designs so that the fiber stress will be either one-fourth the ultimate strength, or the yield point of the material used. If he is an independent engineer he may use one-fourth the elastic limit of the steel; but if he is employed by a company selling material, and is furnishing the plans for nothing, he may use one-fourth the ultimate strength.

Examples of Faulty Methods in Design

The writer recently checked plans for a reinforced-concrete building where the architect had specified that the safe live-load on each floor was to be 125 lb. per sq.ft., with a factor of safety of four; no fiber stresses given in steel or concrete. He stated that when completed a panel on each floor would be selected at random and loaded with four times the combined dead- and live-loads, under which test it should deflect not to exceed $1/_{360}$-th. of the span. Five plans were submitted and in three the steel fiber stress was 20,000 lb. per sq.in., in one it was 18,000 lb. and in the last it was 22,000 lb.; yet each designer stated in his bid that the steel stress was 16,000 lb. The concrete stress was stated to be 700 lb. per sq.in., and it was actually a trifle over 800 lb. in two designs, 900 lb. in two designs, and 1040 lb. in one design. In checking the plans, a ratio of deformation of 15 was used. It was found that some designers had used 8 and some 10, but not one used the ratio fixed by common custom, the action of a number of national societies and by the majority of building ordinances.

Nearly every beam in reinforced concrete is a T-beam, the stem being comparatively narrow and containing only enough concrete to protect the steel and take care of shear, assisted sometimes by stirrups. The flange or upper part of the beam, that takes all the bending compression, extends into the floor slab on either side of the beam. Assume such a beam 24 in. wide in the flange and 12 in. in the stem, with a span of 19 ft. c. to c. of beams. To obtain the load on the slab we deduct the stem thickness from the span, giving 18 ft., and multiply this by the load per lin.ft. (say 100 lb.); obtaining 1800 lb. total load. The bending moment will then be $1800 \times 19 = 34{,}200$ in.-lb. The writer knows from actual experience in checking "free" designs that many men will design as follows: The flange is 2 ft., so that load is $19 - 2 = 17 \times 100 = 1700$ lb. (taking the clear spa nbetween the flange edges). Using the same span the moment (1700×17) is 28,900 in.-lb. In this method, to save steel and thus obtain a contract, the load on the overhanging portions of the flanges is neglected in designing the floor slab, and a condition of perfect restraint is assumed, with no attention paid to any possible bending of the flanges. When such questionable methods are used in competitive designing, it is not strange that a disinterested engineer will find stresses running 10 to 20% higher than the specifications mentioned, although every designer claims to have followed the specifications. It might be thought that the steel salesman would

*Abstract of a paper presented at the annual meeting of the Illinois Society of Engineers and Surveyors in Chicago, Jan. 22-24, 1913.

†Consulting Enginer, Chicago.

try to put in more than enough steel. This he will do if he can arrange to have his steel specified to the exclusion of other steel, but if compelled to compete with other companies it is necessary to cut the steel to the lowest possible limit.

Relations of Steel and Concrete

The stresses should be so balanced that the steel and concrete will be balanced. Studies made to determine the economical proportions are not always satisfactory, for prices vary so greatly for all materials. The writer, however, plotted the result of a number of studies and found the following expression useful in fixing the amounts of steel and concrete. Let fc be unit concrete stress; fs, unit steel stress; n, ratio of deformation. Then $fc = fs \div 1.8\ n$. By fixing the stresses in accordance with the above expression the maximum economy will be obtained. For average prices of materials the economical stresses seem to be practically 500 lb. per sq.in. for concrete in compression and 15,000 lb. for steel in tension with $n = 15$. In spite of this it is not uncommon to see men using a concrete stress of 800 lb. and a steel stress of 20,000 lb.

A 1:2:4 concrete is assumed to have an ultimate strength not far from 2000 lb. per sq.in. in compression and the ratio of deformation is generally assumed to be 15. The safe fiber stress recommended by the Joint Committee is 650 lb. per sq.in. The writer, however, has examined test reports on concrete made in many parts of the United States and finds that such concrete seldom exhibits an ultimate strength exceeding 1800 lb., the strength being fixed by the character of the aggregate, modified by the care used in proportioning and mixing. Poor aggregate may often be used successfully provided the mixing is properly done, so the secret of making good concrete lies in proper mixing after due care has been taken to proportion the material.

Hand mixing should be positively prohibited for reinforced concrete and only batch mixers permitted, except when positive proportioning devices are attached to the mixers. The cement should be always added by weight and not by bulk, the sand and aggregate to be added by bulk, loose. As a rule a 1:2:4 concrete made near Chicago will be stronger than that made near Pittsburgh, owing to the difference in the stone used. The best way to prepare a specification is to state that the fiber stress of the concrete in compression shall not exceed one-third the ultimate strength shown by a test made on a cylinder 8 in. in diameter and 16 in. long, poured at the time the beam or slab is poured and stored under similar conditions to those under which the construction is cured; this strength to be shown at the end of 30 days.

The steel strength is governed by the yield point, which is something different from the elastic limit and should not be confused with it. Twisting steel cold does not raise the elastic limit, which may in fact be slightly lowered by the twisting, but it does raise the yield point considerably. When a steel is stressed past the yield point the dstruction of a reinforced-concrete structure is assured and failure begins. The unit fiber stress used in the steel should not exceed half the yield point for a medium openhearth steel having a yield point in the neighborhood of 33,000 lb. per sq.in., provided mechanical bond is provided. Without a mechanical bond the unit stress should not exceed 12,000 lb., for the adhesion is weakened by continued dryness, or long continued submergence, as in tanks. For steel having a yield point exceeding 40,000 lb. the unit fiber stress may be one-third the yield point, but should never under any circumstances exceed 18,000 lb. Without a mechanical bond, the steel stress should never exceed 12,000 lb., no matter how strong the steel. Rerolled steel may be used safely if it complies with the standard specifications of the steel manufacturers association.

Cement is a very uniform material when purchased from the larger companies, yet it is the almost universal practice to pay for cement testing, while steel is bought on faith. The man who neglects to have his steel tested is taking unjustifiable chances. The writer, if governed by small appropriations, would rather use cement without testing than steel without testing, provided that the cement is provided by a manufacturer of good reputation. The majority of men selling steel are merely brokers under contract to dispose of the product of mills rolling (not necessarily manufacturing) steel. In all specifications there should be a clause requiring that the steel weigh within 5% of the standard weights given in steel manufacturers' handbooks. Unfortunately much light steel is marketed.

Columns

In reinforced-concrete columns it is good practice to specify that the steel be set so that a cover of not less than 2 in. of concrete will be provided. This is for fire protection, and many engineers will not consider any of the concrete outside the steel as carrying load. Thus if the design requires a 12-in. column, the fire protection will cause the use of a column 16x16 in. The writer has never yet examined a competitive design in which the concrete outside the steel was excluded. In his own practice he adds one inch each way to the dimension of the column for fire protection, for it is hardly likely any fire will injure the concrete to a great depth. The steel, however, is set 2 in. from the faces. Concrete increases in strength with age and on an average-sized column a reduction in dimension of one inch each way after the expiration of one year will not make the column any weaker than it was the day it was first put in use.

Columns of the hooped variety always show signs of failure by the concrete peeling away from the steel, so for such columns it is necessary to consider only the concrete area inclosed within the inner face of the hooping. Competitive designers may use the full area of the concrete in computing the strength. The concrete stress for hooped columns being much higher than for columns vertically reinforced without hooping makes this sort of designing highly objectionable.

Improved Methods in Design

True economy and proper service in reinforced-concrete design can be obtained only by having the design prepared by a man paid for his work, and not tied to any steel or any system of reinforcement. The reputable companies, however, should not be prevented from helping the architect or engineer by giving him the benefit of their experience. So every specification should contain clauses giving design-standards, and competition should be permitted, provided the lowest bidder with his own design will consent to a checking of his design (he to pay for the checking).

These standards should be stated plainly in the specifications, and the writer recommends the following clauses:

> Parties wishing to submit competitive designs for this structure may do so provided said designs conform in every detail to the standard building specifications for reinforced-concrete structures adopted by the National Association of Cement Users, the floor loads being as follows. . . . .

In asking for plans it is not sufficient to require a factor of safety or to simply mention steel stresses. The factor of safety should not be mentioned. When steel stresses are given it is necessary also to fix the ratio of deformation. It is positively necessary to state the bending-moment coefficients and define the span lengths. When this is done the amount of steel to be placed over supports, as compared with the steel in the middle, must be mentioned, and the amounts should be equal if the middle positive coefficient is $1/12$. The area of the concrete to be considered in column design must be fixed; also the percentage of live-load on each floor going to the columns.

These are a few of the things to be considered and the specifications mentioned above should be carefully studied to insure a design according to the intention of the architect, engineer or owner.

Delay in Awarding the Mechanical Filtration Contract for the Croton Water-Supply, New York City, has been secured by the Bureau of Municipal Research and the Citizens' Union, of New York City, acting through Comptroller Wm .A. Prendergast, a member of the Board of Estimate and Apportionment. By agreement, the bids were opened on Jan. 23, with the understanding that the award would be postponed until the subject could be looked into further. A pamphlet of 129 pages, reviewing both the hold-up and the history of the preparation of the filtration plans, and also reprinting a report (June 10, 1912) of a board of engineering experts on the plans, has been published for general circulation by the Department of Water Supply (Park Row Building, New York City).

The detailed bids for this work were published on p. 48 of our Construction News section of Jan. 30, 1913. By a typographical error the total bid of the Keystone State Construction Co. (2 Rector St., New York City) was wrongly given, both in the table and in a brief news note on p. 236 of the same issue. The correct figure was $5,139,015, which was the lowest bid received. The lowest bid of the highest bidder was $6,752,500. The estimate of I. M. de Varona, chief engineer of the Department of Water Supply, was $5,916,700. The lowest bid was 13% lower than the engineer's estimate, and the highest bid (really the lowest bid of the highest bidder) was 14% higher than the engineer's estimate, while the average bid was only 1½% higher than the engineer's estimate. We expect to give some further details of the bids in our Construction News section for Feb. 13.

Following a Cattle-train Derailment near Cairnes, Queensland, Australia, a passenger express train ran into the wreckage and killed six and injured five other persons. It is said that this is the first serious accident on the Queensland railways in years.

A Collision Occurred on Feb. 2, between the Hamburg-American liner "Prinz Oskar" and the four-masted schooner "City of Georgetown," in which the former vessel was badly damaged and the latter was sunk. The scene of the accident was the Five Fathom Bank Light beyond the Delaware Breakwater. No lives were lost as a result of the collision.

The Proposed Lincoln Memorial Road from Washington, D. C., to Gettysburg, Penn., was effectually killed last week by the passage through Congress of a bill which authorized the appropriation of $2,000,000 for a Lincoln memorial in Washington, to take the form of a suitable building, statute and grounds in the river park, which will fit into the general development of the city plan recommended by the McKim Commission. The proposed road, which was very pertinaciously advocated by the federal road aid enthusiasts, was to be a broad boulevard between the national Capital and the Gettysburg battlefield, and was to be built and maintained by the federal government. Pres. Taft signed the bill on Feb. 1.

A Panic Broke Out in a moving-picture theater in New York City, on Feb. 2, when one of the films took fire. The operator extinguished the flame, but the audience of over a thousand fled in a wild panic in which two persons were killed and between 20 and 30 injured. The fatalities occurred at a short flight of steps leading from the theater to the side walk. People fell on the steps and others were crowded upon them until exit from the theater was entirely blocked. The dead were found at the bottom of this heap of persons. PERSONALS N. R.

A Society for the Greater Development of the electrical industry at large has been incorporated in New York City under the name of the Society for Electrical Development. The aim of the society is to increase use of electricity, especially for light, heat and power. Four branches of the industry are represented by men from prominent concerns therein; (1) electric light and power companies, (2) manufacturers of electrical apparatus, (3) electric supply jobbers, (4) electrical contractors. H. L. Doherty has been elected President; J. M. Wakeman, General Manager; and P. F. Dodd, Secretary-Treasurer.

An Appeal from the Chicago Drainage Canal Decision rendered by Secretary of War Stimson, on Jan. 8, 1913, denying the application of the Trustees of the Sanitary District of Chicago for permission to increase the amount of water diverted from Lake Michigan to and through the Canal, seems likely. It has not been announced whether the appeal will be made to Congress, as suggested in the course of editorial comment, which accompanied our reprint of the decision in our issue of Jan. 13, or to the new Secretary of War, after Mar. 4. It seems likely, however, that the appeal may be to the latter. However this may be, the trustees do not seem to be willing to accept the decision of Secretary Stimson as final. Their conclusion seems to be based partly on legal points and partly on what the trustees believe to be important new evidence bearing upon the case.

We have received from Lyman E. Cooley, the recently appointed consulting engineer to the trustees of the Sanitary District, a statement bearing upon this subject which we condense as follows: After an interview with Gen. Wm. H. Bixby, Chief of Engineers, U. S. Army, Mr. Cooley wrote to the Secretary of War, on Dec. 30, 1912, requesting permission to withdraw or amend the application in question and stating that within two weeks the trustees of the Sanitary District would be able to present in full their reasons for the request. In an interview held subsequently, Secretary Stimson expressed unwillingness to reopen the case or to defer his decision, but stated that he would hold his findings for a few days and that meanwhile he would consider citations on certain legal propositions. On Dec. 31, a statement presented before the Senate Committee on Commerce (Apr. 16, 1912) on behalf of the city of Chicago was left at the Secretary's office. On Jan. 6, 1913, M. B. Madden interviewed the President and the Secretary of War with a view to having the decision deferred until the additional or supplementary matter could be presented. The subject was considered also at a cabinet meeting on Jan. 7, when further time for consideration was asked by the Secretary of the Interior (Mr. Fisher) and by the Secretary of the Treasury (Mr. MacVeagh). The attorney of the Sanitary District of Chicago went to Washington to present statements prepared by him and by the consulting engineer, but on his arrival (Jan. 10) he was refused an interview and was told that no further briefs or arguments would be considered or would change the adverse decision which had already been rendered (Jan. 8). In his letter accompanying the unaccepted statements, the consulting engineer had explained that the officials of the Sanitary District were preparing additional matter in support of their application, which they considered important, and Mr. Cooley informs us that this matter is now undergoing revision and will be made public in the early future.

At the meeting of the Trustees of the Sanitary District, held on Jan. 23, a report was submitted by the Trustees' Special Commission on sewage disposal and water-power development ("Eng. News," Mar. 7, 1912). The main statements of the report were as follows:

After a careful investigation of the effect upon lake levels and the legal and legislative phases involved, your Commission does not believe that the government will ultimately prevent the utilization of the main drainage canal to the full purpose for which it was designed.

As the ultimate capacity of the main drainage channel is 14,000 cu.ft. per sec., and at the necessary legal rate of flow of 3¼ cu.ft. per sec. per 1000 of tributary people the capacity of the canal will serve for a population of over 4,000,000, we advise that (pending the submission of the final report of this commission) no departure from the dilution method of sewage disposal, as provided for by law, should be contemplated by the Sanitary District, except for treatment of trade wastes and special effluents.

It is interesting to note that the International Waterways Commission, in its report of Jan. 4, 1907, on the Chicago Drainage Canal, recommend that the government of the United States "prohibit the diversion of more than 10,000 cu.ft. per sec. for the Chicago Drainage Canal." Secretary Stimson was far less liberal when he refused to allow an increase of the present diversion of 4157 ft.

PERSONALS

Mr. L. J. Houston, Jr., Assistant Chief Engineer of the Paving Commission of Baltimore, Md., has resigned.

Mr. Seton Porter, for the past eight years associated with Sanderson & Porter, Engineers and Contractors, New York City, has been admitted to membership in the firm.

Mr. Allen J. Saville, for the past three years Assistant City Engineer of Richmond, Va., has resigned to engage in private practice in partnership with Mr. Burton Marye, with offices in Richmond.

Mr. Lester W. Tucker, M. Am. Soc. C. E., formerly with Westinghouse, Church, Kerr & Co., New York City, has been appointed General Manager of the Brinson Ry. Co., with headquarters at Savannah, Ga.

Mr. Percy R. Todd, recently Vice-President, has been elected President of the Bangor & Aroostook R.R., in Maine. Mr. Todd was formerly Vice-President of the New York, New Haven & Hartford R.R., at New Haven, Conn.

Mr. E. C. Blundell, formerly Assistant Superintendent of the Chicago, Milwaukee & St. Paul Ry., at Eau Claire, Wis., has been promoted to be Superintendent of the Nebraska division at Omaha, Neb., succeeding Mr. J. R. Welch, transferred to St. James, Minn.

Mr. E. T. Adams, M. Am. Soc. M. E., former Manager and Chief Engineer of the gas and mill engine department of the Allis-Chalmers Co., Milwaukee, Wis., and recently President of the Wisconsin Engine Co., Milwaukee, has been appointed Chief Engineer of the Rumely Co., La Porte, Ind.

Mr. W. L. Saunders, M. Am. Soc. M. E., President of the Ingersoll-Rand Co., New York City, has been elected President of the International Harvester Corporation, recently organized to handle the foreign business of the International Harvester Co., of Philadelphia, Penn. Mr. Saunders will continue as President of the Ingersoll-Rand Co.

Mr. Robert I. Randolph, Assoc. M. Am. Soc. C. E., has resigned as Secretary of the Rivers and Lakes Commission of Illinois. He will continue to attend to such matters as may come before his department until a successor is chosen. The position will be filled by a civil-service examination, the announcement of which will be made later, and it is desired to obtain a civil engineer. The salary is $300 per month.

Mr. Fletcher M. Hamilton has been appointed State Mineralogist of California, succeeding Mr. W. H. Storms, who was summarily removed by Governor Johnson without other cause than that he was not in harmony with the administration. Mr. Hamilton is a graduate in mining engineering of the University of California, class of 1904. He has had practical

mining experience in California, Nevada, Montana and Mexico.

Mr. D. L. Bush, recently General Manager of the Chicago, Milwaukee & St. Paul Ry., has been elected Vice-President with headquarters at Chicago, Ill. Mr. Bush has passed through all the positions in operating department from night telegraph operator in a backwood's town to Superintendent, General Superintendent, and in 1909 to General Manager. He was born in 1855 and has been in the railway service since he was 17 years old.

Mr. Howard D. Bennett, Consulting Engineer, of Baltimore, Md., formerly in charge of the designing department of the Ellicott Machine Co., is now associated with the Skinner Ship Building & Dry Dock Co., of Baltimore, as Engineer in charge of the dredge department, which has recently been added to the company's business. Mr. Bennett will also continue his private practice as a consulting engineer.

Mr. Virgil G. Bogue, M. Am. Soc. C. E., former Chief Engineer of the Western Pacific Ry., has recently been engaged in an economic study of line improvement and double-tracking of the Canadian Pacific Ry. through the Selkirk and Rocky Mountains in British Columbia. Mr. A. W. Buel, M. Am. Soc. C. E., who has been associated with Mr. Bogue in this work, has returned to his consulting practice, at 15 William St., New York City, after an absence of three months.

Mr. E. Warren Ritchie, formerly with the Purdy Engineering Co., in charge of a large hydro-electric development in Costa Rica, has opened an office in Petersburg, Va., as a consulting engineer in hydro-electric and municipal engineering work. Mr. Ritchie is a graduate of the Massachusetts Institute of Technology, class of 1898. He had charge of the surveys in Havana, Cuba, upon which the present sewerage system is based. Mr. Ritchie also made the preliminary surveys for the Jamestown Exposition.

Mr. Charles H. Lee, Assoc. M. Am. Soc. C. E., for several years Assistant Engineer on the Los Angeles, Calif., water-supply aqueduct, in charge of water-supply investigations in Owens Valley, has opened an office at 934 Union Oil Bldg., Los Angeles, as a consulting, civil and hydraulic engineer. Recently Mr. Lee has carried on underground water investigations in various parts of central and southern California for the State Conservation Commission, and has also served as Hydraulic Engineer for the Cuyamaca Water Co., of San Diego.

Mr. Richard S. Buck, M. Am. Soc. C. E., has retired from membership in the firm of Sanderson & Porter, Engineers and Contractors, New York City, to become Chief Engineer of the Dominion Bridge Co., Montreal, Que. Mr. Buck graduated from Rensselaer Polytechnic Institute in 1887. He was Resident Engineer during the construction of the famous Niagara River steel arch bridge; the Williamsburg bridge, East River, New York, was built under his supervision, and for a number of years he was Chief Engineer of the Department of Bridges of New York City.

Mr. E. H. McHenry, M. Am. Soc. C. E., who for the past two years has been Vice-President in charge of maintenance and construction of both the New York, New Haven & Hartford R.R. and the Boston & Maine R.R., will hereafter have charge of these departments of the New York, New Haven & Hartford R.R. only. Mr. H. J. Horn, whose retirement from the New Haven system was noted in our issue of last week, will have charge of these departments of the Boston & Maine R.R., as well as the operating department.

Mr. Miles Bronson, Superintendent of the Electric division of the New York Central & Hudson River R.R., has been promoted to be General Manager of the Grand Central terminal, New York City, in addition to his other duties, succeeding Mr. A. R. Whaley, resigned as noted in our issue of last week. Mr. Bronson was born in British India in 1875. His railway work began in 1890 in the law department of the Grand Trunk Ry. He has been private secretary to the President, Assistant to the President and Division Superintendent with the New York Central Lines.

Mr. H. B. Earling, formerly Assistant General Manager of the Chicago, Milwaukee & St. Paul Ry., has been elected Vice-President, with headquarters at Seattle, Wash., succeeding Mr. H. R. Williams, transferred to New York City. Mr. Earling entered the service of the Chicago, Milwaukee & St. Paul Ry. in 1878, as an agent and telegraph operator. He was promoted through the grades of assistant train dispatcher, chief train dispatcher, Trainmaster, Superintendent of Terminals, Division Superintendent, Assistant General Superintendent and General Superintendent to that of Assistant General Manager about a year ago.

Mr. E. D. Sewall, formerly Assistant to the President of the Chicago, Milwaukee & St. Paul Ry., has been elected Vice-President, with headquarters at Chicago, Ill. Mr. Sewall was born in Wilmington, Del., in 1855. At 16 years of age he entered the service of the Wicomico & Pocomoke R.R. as a rodman. In 1872 he went West and spent 10 years in the engineering departments of the Northern Pacific Ry., the Chicago & Canada Southern Ry., the St. Paul, Minneapolis & Manitoba Ry. and the St. Paul & Duluth R.R. From the engineering he changed to the transportation branch of the railway service as a clerk of the general freight department of the St. Paul & Pacific Ry., in 1881. Later he was Assistant Engineer and clerk to the General Superintendent of the St. Paul & Duluth R.R. For a few months in 1882, he was joint agent of the St. Paul & Duluth R.R. and the Chicago, Milwaukee & St. Paul Ry., at Stillwater, Minn.; and then for five years he was agent for the latter road at the same place. After nearly two years' experience as traveling lumber agent he was appointed, in 1889, commercial agent of the Chicago, Milwaukee & St. Paul Ry., at St. Paul, Minn. In 1895 he was appointed Assistant General Freight Agent of the Superior division, at Milwaukee, Wis.; and for four years, 1898 to 1902, he was Northwestern Agent at Minneapolis, Minn. Mr. Sewall was promoted to be Assistant General Superintendent of the Chicago, Milwaukee & St. Paul Ry. in 1902, and in 1906 was appointed Assistant to the President. He was Vice-President of the Chicago, Milwaukee, Puget Sound Ry., from 1909 to the recent consolidation of this road with the older company.

Mr. Edward E. McCall, a Supreme Court Justice of New York, has been appointed Chairman of the Public Service Commission, First District, of the State of New York, succeeding Mr. William R. Willcox, who has been Chairman since the establishment of the commission during Governor Hughe's administration. The Chairman of this Public Service Commission is the executive head of probably the largest engineering organization in the world, which is about to supervise the construction of subway and rapid transit lines in Greater New York to cost many millions of dollars. Mr. McCall graduated from New York University Law School in 1884, and practiced law in New York City until 1902, when he was elected a justice of the state supreme court.

Mr. W. W. Colpitts, M. Am. Soc. C. E., has resigned his position of Chief Engineer of the Kansas City, Mexico & Orient Ry., to become associated with W. H. Coverdale & Co., Consulting Engineers, New York City. He will, however, continue to be Chief Engineer of the company's lines in Mexico. Mr. Colpitts is a graduate in civil engineering of McGill University, class of 1899, and began his engineering experience as a draftsman in the office of the Chief Engineer of the Intercolonial Ry. He remained in the engineering departments of various Canadian railways until 1899, when he was appointed Chief Clerk in the office of the President of the Canadian Pacific Ry., at Montreal, Que. In 1900, he returned to engineering work as Assistant Engineer in the construction department. A year later he joined the enginering staff of the Kansas City, Mexico & Orient Ry. as Assistant Engineer and was promoted to be Division Engineer, and in 1909 to be Chief Engineer.

OBITUARY

Thomas J. O'Connor, a young civil engineering employee on the construction of the plant of the Air Turbine Co., Harrison, N. J., was killed, Jan. 31, by a fall from an erecting tower. Mr. O'Connor was a native of New Haven, Conn., and a recent graduate of Sheffield Scientific School, Yale University.

John Harlan Freeman, M. Am. Soc. M. E., a consulting mechanical engineer with offices at 45 Broadway, New York City, died at his home in Brooklyn, N. Y., Jan. 27. He was born in Michigan 45 years ago, and was a graduate of the Michigan Agricultural College. For a number of years he was in the U. S. Patent Office, Washington, D. C. He came to New York City in 1890.

A. H. Coon, of Kingston, Penn., for many years a well known contractor, died Jan. 31, at the age of 84 years. He was still active in local contract work after his 83d year. Mr. Coon was contractor for more than 40 complete waterworks. He was first to lay rails for street-car service in the State of Pennsylvania, which was on a line built in 1856, between Kingston and Wilkes-Barre.

Mrs. Lavinia Marston Swasey, wife of Ambrose Swasey, of Cleveland, Ohio, Past-President of the American Society of

Mechanical Engineers, died at her home in Cleveland, on Jan. 22, after a very brief illness. Mrs. Swasey was born at Hampton, N. H., July 28, 1842. She had been a resident of Cleveland for more than thirty years and had taken a prominent part in charitable and philanthropic work.

James B. Hammond, President of the Hammond Typewriter Co., and inventor of one of the first practical typewriters, died Jan. 26, at St. Augustine, Fla., while on a yatching trip. He was born in Boston, Mass., Apr. 23, 1839, and graduated from the University of Vermont in 1861. He studied for the ministry, served during the Civil War as a war correspondent for the New York "Tribune"; and after numerous experiments patented his typewriter in 1880. During the past few years he had been noted chiefly for his eccentricities.

Gaylord M. Beach, former Assistant General Manager of the Pittsburgh & Lake Erie R.R., died at his home in Pittsburgh, Penn., Jan. 28, aged 81 years. He was retired on a pension, two years ago, after 61 years of active railway service, begun in 1850 as a track laborer on the Sandusky, Mansfield & Newark R.R. From track laborer he was promoted to the position of foreman of construction gangs and laid many miles of railway track in the Middle West and South in the decade preceding the Civil War. He then became a conductor of freight and passenger trains, Fuel Agent and Assistant Roadmaster of the Cleveland, Columbus, Cincinnati & Indianapolis Ry., and in 1869 Superintendent of Construction of the Indianapolis & St. Louis Ry. For a short time he was Roadmaster of the Atlantic & Pacific R.R. and the South Pacific R.R. in Missouri. Following January, 1871, he served 8 years as Real Estate and Tax Agent of the Cleveland, Columbus, Cincinnati & Indianapolis R.R., and two years as Roadmaster of the Indianapolis division of the same railway. From 1880 to 1885 he was in charge of the engineering department of this railway, which was combined with the Indianapolis & St. Louis Ry. in 1882. From 1885 to 1889 he was General Manager of the combined railways. In 1890 Mr. Beach went to the Pittsburgh & Lake Erie R.R. as General Superintendent. Eight years later he was promoted to be Assistant General Manager. In 1900 he retired to the less active position of Special Agent at Pittsburgh.

ENGINEERING SOCIETIES

COMING MEETINGS

MAINE SOCIETY OF CIVIL ENGINEERS.
Feb. 10-11. Annual meeting at Augusta, Maine. Secy., F. E. Pressey, 487 Union St., Bangor, Maine.

CONNECTICUT SOCIETY OF CIVIL ENGINEERS.
Feb. 11-12. Annual meeting at New Haven, Conn. Secy., J. F. Jackson, New Haven, Conn.

MINNESOTA SURVEYORS AND ENGINEERS SOCIETY.
Feb. 11-13. Annual convention at St. Paul, Minn. Secy., Charles A. Forbes, 91 Kent St., St. Paul, Minn.

OREGON SOCIETY OF ENGINEERS.
Feb. 13. Annual meeting at Portland, Ore. Secy., J. C. Stevens, 605 Spalding Bldg., Portland, Ore.

NORTH DAKOTA SOCIETY OF ENGINEERS.
Feb. 13-14. Annual meeting at Bismarck, N. D. Secy., E. F. Chandler, University, N. D.

NEW ENGLAND ASSOCIATION OF GAS ENGINEERS.
Feb. 19-20. Annual meeting at Boston, Mass. Secy., N. W. Gifford, Boston, Mass.

IOWA ENGINEERING SOCIETY.
Feb. 19-21. Annual meeting at Sioux City, Iowa. Secy., S. M. Woodward, Iowa City, Iowa.

IDAHO SOCIETY OF ENGINEERS.
Feb. 21-22. Annual meeting at Weiser, Idaho. Secy., I. F. Shaffner, Boise, Idaho.

ASSOCIATION FOR STANDARDIZING PAVING SPECIFICATIONS.
Feb. 24. Annual meeting at Pittsburgh, Penn. Secy., J. B. Hittell, 5917 Winthrop Ave., Chicago, Ill.

INDIANA SANITARY AND WATER SUPPLY ASSOCIATION.
Feb. 25-26. Annual meeting at Indianapolis, Ind. Secy., W. F. King, Indianapolis, Ind.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
Feb. 26-28. Midwinter Convention in New York City. Secy., F. L. Hutchinson, 33 W. 39 St. New York, N. Y.

CENTRAL ELECTRIC RAILWAY ASSOCIATION.
Feb. 27-28. Annual meeting at Indianapolis, Ind. Secy., A. L. Neereamer, Indianapolis, Ind.

NATIONAL PAVING BRICK MANUFACTURERS ASSOCIATION.
Mar. 3-5. Annual meeting at Chicago, Ill. Secy., W. P. Blair, 824 Engineers Bldg., Cleveland, Ohio.

ILLINOIS WATER SUPPLY ASSOCIATION.
Mar. 11-12. Annual meeting at Urbana, Ill. Secy., Edward Bartow, Urbana, Ill.

BOSTON SOCIETY OF CIVIL ENGINEERS.
Mar. 19. Annual meeting at Boston, Mass. Secy., S. E. Tinkham, 715 Tremont Temple, Boston, Mass.

UTAH SOCIETY OF ENGINEERS.
Apr. 18. Annual meeting at Salt Lake City, Utah. Secy., R. B. Ketchum, 702 Newhouse Bldg., Salt Lake City, Utah.

Kansas Engineering Society—The feature of the annual meeting at Topeka, Kan., on Jan. 21-22, was an address by Hugh L. Cooper on "The Keokuk Dam." Among the other papers on the program were the following: "Construction Cost Data," by F. D. Hughes; and "The Value of Pure Water," by G. R. Jones.

The officers elected for the ensuing year were: President, J. M. Meade; vice-president, H. A. Rice; secretary-treasurer, T. J. Strickler; directors, R. V. Leeson, J. W. Mavity, Fred Hesser and H. A. Rowland.

Maine Society of Civil Engineers—At the annual meeting at Augusta, Maine, on Feb. 10-11, a paper will be read on "The Use of the Pitometer," by Edward G. Lee. Secretary, Frank E. Pressey, Bangor, Maine.

Connecticut Society of Civil Engineers—Among the papers on the program for the annual meeting at New Haven, Feb. 11-12, are the following: "Relation of the Engineer to Public Health," by F. H. Snow; "The Chemist and Physicist in Relation to the Engineer," by W. R. Whitney; "The Catskill Aqueduct of the New York City Water Supply," by Clarence Blakeslee; and "Electrification on the New York, New Haven & Hartford Railroad," by W. S. Murray.

American Institute of Electrical Engineers—It has been decided to have the midwinter convention of the Institute, in New York City, Feb. 26-28, a working convention, so no excursions or other special entertainment features have been planned, except the reception and dance at the Hotel Astor on the evening of Feb. 28.

An extensive technical program has been prepared, containing papers under the head of "Heating, Heat Measurements and Rating by Heat," "Methods of Determining Losses in Apparatus," "Methods of Testing Apparatus for Performance," "Miscellaneous Subjects Relating to Rating." Particular interest will center about the paper on "Temperature and Electrical Insulation," by C. P. Steinmetz and B. G. Lamme (Sub-committee on Revision of Rules) and that on "Method of Rating Electrical Apparatus," by W. L. Merrill, W. H. Powell and Charles Robbins (Sub-commitee on Rating). The morning session on Feb. 27 will open with an address by H. G. Stott on the use of the "myriawatt" and "myriawatt-hour" terms proposed by him at the last annual convention as substitutes for the boiler horsepower and boiler horsepower-hour.

Albany Society of Civil Engineers—The ninth annual meeting of the society was held at Keeler's Hotel, Albany, N. Y., Jan. 28. The report of the secretary aroused much enthusiasm and gave encouraging evidence of the growth and influence of the society. During the past year the membership has increased from 110 to 216, and includes many engineers prominent in the state barge canal and highway work and in the federal service. After a short business session the members proceeded to the banquet hall to enjoy their annual dinner. The toastmaster was Richard W. Sherman, Chief Engineer of the State Conservation Commission. Among the after-dinner speakers were Robert E. Horton, Consulting Hydraulic Engineer, of Albany, who was for many years in charge of the water resources branch of the U. S. Geological Survey in New York State; Arnold D. Chapman, Deputy State Engineer of New York; P. H. Parthesius, Senior Engineer Examiner of the New York State Civil Service Commission, of Troy, N. Y.; John A. O'Connor, Engineer of Terminals, New York State Barge Canal; C. C. Covert, District Engineer, U. S. Geological Survey in charge of water resources surveys, in New York State; F. P. Lanigan, City Engineer, of Albany; A. C. Harper, Assistant Engineer, U. S. Engineer Office, Troy, N. Y., and Mortimer G. Barnes, Consulting Engineer of the New York State Barge Canal. The officers elected for the ensuing year are: President, William R. Hill, Consulting Engineer, of Albany; Secretary, Rupert Sturtevant, Assistant Engineer, New York State Barge Canal; Treasurer, E. H. Sargent, Assistant Engineer, State Conservation Commission.